CITY OF LODI
INFORMAL INFORMATIONAL MEETING
"SHIRTSLEEVE" SESSION
CARNEGIE FORUM, 305 WEST PINE STREET
TUESDAY, SEPTEMBER 19, 2000

An Informal Informational Meeting ("Shirtsleeve" Session) of the Lodi City Council was held Tuesday, September 19, 2000 commencing at 7:05 a.m.

A. ROLL CALL

Present: Council Members – Hitchcock, Land, Nakanishi, and Pennino

Absent: Council Members – Mayor Mann

Also Present: City Manager Flynn, Deputy City Attorney Schwauber and City Clerk Blackston

B. CITY COUNCIL CALENDAR UPDATE

City Clerk Blackston reviewed the Mayor's and Council Member's Weekly Calendar (filed).

C. TOPIC(S):

1. "Wastewater Treatment Plant Master Plan"

Public Works Director Prima recalled that a number of months ago staff advised Council the State had issued a revised permit that would significantly change how wastewater at the White Slough Water Pollution Control Facility would be managed. In response, a public advisory committee was formed. Mr. Prima reported that the plant is now 30 years old and improvements are needed. The new permit requires the City to go to tertiary treatment, which is an advanced form of filtration and a higher level of disinfection. Currently, secondary treatment is provided at the facility and surface discharge is done at Dreger Cut. Mr. Prima briefly reviewed possible alternatives.

Bruce West, Project Manager with West Yost & Associates, explained that they only recently received information from the Regional Board about discharge requirements and other water studies which have impacted completion of their work. Mr. West submitted copies of the Wastewater Master Plan and overheads (filed). By the end of the 20-year Master Plan (estimating a 1.5 percent growth rate) the population of Lodi will approach 80,000 and flows at the wastewater plant would be 8.5 million gallons per day (mgd).

Mr. West reported that minimal growth is anticipated for industrial dischargers. The types of improvements to accommodate these flows would be relatively minor compared to the municipal improvements that are required at the plant and for disposal. He added that the major industrial dischargers are doing water conservation work.

Council Member Hitchcock questioned the population estimate of 1.5 percent, noting that the City's average over a 30-year period was determined to be 2 percent.

Mr. Prima replied that if population increased 2 percent during the next twenty years, capacity at the plant would be reached in 2015 rather than 2020.

Mr. West commented that in his experience with a recent $60 million project, connection fees were a significant part of financing. He explained that if growth occurs faster than anticipated, there will be more connection fee money to do the next expansion; and if growth is slower, it will delay the need for expansion.

Mr. West stated that the current discharge location is at Dreger Cut, which extends from the edge of the City property out into the Delta and terminates at Bishop Cut. He reported that Bishop Cut has higher flows and better dilution. Rather than discharging into the Delta, Mr. West described an alternative of applying all or part of the wastewater to land and percolating it into the ground. He noted that the land east of I-5 has higher permeable soils.

Rob Beggs, Project Engineer with West Yost & Associates, reported that with the new permit requirements, the City will not be able to discharge into Dreger Cut when the dissolved oxygen level drops below 5 milligrams per liter. He explained that this sometimes occurs independently of whether or not the City is discharging there. Data indicates that Bishop Cut maintains much higher levels of dissolved oxygen. Temperature change is also addressed in the new permit requirements. Discharging into Dreger Cut during the winter would increase the temperature more than the limit. Calfed is proposing to open some Delta channels to let more water flow from north to south through the Delta. If this is done, it will reduce some of the net flows through Bishop Cut.

Mr. Beggs reviewed the subjective evaluation criteria, which was established with input from the public committee. He explained that benefits from the constructive wetlands option include storage, the reduction of temperature and nitrates, and the removal of heavy metals and trace toxic constituents.

Mr. Beggs outlined improvements that need to be made. He explained that wastewater comes into the headworks at the treatment plant. There is need for improvement in the headworks, pumping, screening, and odor control. By the year 2020 the headworks will need to be replaced. Minor improvements in the septage handling facilities are also recommended. Currently, there are four basins for secondary treatment. Two additional basins are recommended in order to continue nitrifying, which is an implicit condition of the new permit. Clarified effluent now goes to chlorine contact tanks. It is recommended for safety reasons, that gaseous chlorine be changed to liquid chlorine. A third clarifier is recommended both for capacity and redundancy. Filters are required in the new permit requirements if discharge to Dreger Cut is continued. Secondary solids are thickened in dissolved air flotation thickeners. Currently, there is only one thickener, and two are being recommended for redundancy and capacity. A new digester is needed for digesting the solids. Converting one of the old aeration basins to a digested sludge storage basin is recommended. New odor collection, ducting, fans, and biofilter foul air treatment is needed.

In response to Mayor Pro Tempore Nakanishi, Mr. Beggs explained that if the City goes to tertiary treatment, the water can be discharged into the river except when the dissolved oxygen level is too low. If discharge to land is done, tertiary treatment will probably not be required. The current permit requires tertiary treatment by 2004 if discharge to Dreger Cut is continued. For discharge to any other location, a new permit will be required.

Mr. Prima added that tertiary treatment would likely be required for discharge to Bishop Cut as well.

Mr. Beggs stated that Lodi has more zinc in its effluent than most other municipalities. The source has not been identified. He expressed confidence that the zinc requirement (limit of 100 micrograms per liter) could be met by tertiary treatment, or by use of wetlands which would remove it. To provide additional capacity to handle the peak flows at 8.5 mgd, improvements are needed to the following: domestic pumps, industrial pumping site, thickening for the waste activated sludge, sludge lagoon capacity, and a new anaerobic digester.

Mr. Beggs explained that one of the most significant changes in the new regulations is in nitrification (converting ammonia to nitrate). Nitrates now must be removed, which will cut capacity from 8.5 mgd to approximately 6.5 mgd. Mr. Beggs reviewed the alternatives for effluent discharge, reuse and storage. He noted that complete land discharge is an attractive alternative from a regulatory perspective. Currently, there are about 790 net available acres. For alternatives that involve winter percolation and discharge to the Delta, 260 additional acres are needed. For complete land discharge, an additional 400 acres of sandy land is needed. The City currently has sufficient land for industrial wastewater, bio solids, and some municipal wastewater. Additional land is needed for municipal effluent, especially if wetlands and percolation basins are desired.

In summary, Mr. Beggs reported that the three preferred alternatives and associated costs are: 1) Bishop Cut discharge with wetlands, which includes tertiary treatment – $33.8 million; 2) Bishop Cut partial discharge with wetlands, which does not include tertiary treatment – $30.7 million; and, 3) land discharge – $33.9 million.

Mr. West explained that one of the assumptions related to land disposal deals with winter percolation rates and how much wastewater can be applied per acre. He recommended pilot testing of significant areas to test the rates of winter percolation. He also suggested submitting an application to the Regional Board to determine what the discharge permits would be for surface waters. Mr. West explained that regional permits come up for review every five years. He stated that today's presentation is the first of several public participation processes. Final recommendations will be determined at a later date.

Mr. Prima reported that the issue of pilot testing would be brought back to Council for consideration.

Council Member Pennino suggested that consideration also be given to economic development goals, as well as impacts of industrial and residential conservation.

Mayor Pro Tempore Nakanishi requested a summary explaining why water cannot be discharged into the river after tertiary treatment and removal of chemicals.

<u>PUBLIC COMMENTS</u>

a) Wilbert Ruhl spoke in support of land discharge. He warned that regulations may soon prohibit discharging into the Delta. He urged the Council not to allow a sports complex to take up land that should be used for water disposal.

D. <u>COMMENTS BY THE PUBLIC ON NON-AGENDA ITEMS</u>

None.

E. <u>ADJOURNMENT</u>

No action was taken by the City Council. The meeting was adjourned at 8:20 a.m.

ATTEST:

Susan J. Blackston
City Clerk

CITY OF LODI

COUNCIL COMMUNICATION

AGENDA TITLE: Wastewater Treatment Plant Master Plan

MEETING DATE: September 19, 2000 (Shirtsleeve Session)

PREPARED BY: Public Works Director

RECOMMENDED ACTION: None needed; direction on public participation and future Council discussions will be requested.

BACKGROUND INFORMATION: City staff, our consultants - West Yost, and a public advisory committee have been working on a long range plan for meeting wastewater treatment requirements. We have looked beyond our current permit and attempted to assess long-term strategies.

We have reached the point where public involvement is appropriate as we continue to evaluate options. We have two fundamentally different options – one is to provide advanced treatment through various sub-options and continue to discharge to the Delta at our present location – Dredger Cut – or at a new location – Bishop Cut. The second is to acquire additional land for percolation and total land discharge.

These options are summarized on the attached chart. At the shirtsleeve session, staff and our consultants will review the options in more detail and respond to questions and comments. Following this, and additional public input, we will narrow the options and make a recommendation.

Note that estimated costs are shown, and while they vary by a few million dollars, when considered on an annual basis, the difference is relatively small. Also, we are not prepared to discuss financing and rate impacts at this time. The possibility of obtaining grants and the potential impacts of the Pro-Style Sports Facility and the CalPine power plant could greatly reduce the cost.

FUNDING: Wastewater Fund - none needed at this time.

Richard C. Prima, Jr.
Public Works Director

RCP/lm

Attachment

cc: Fran Forkas, Water/Wastewater Superintendent
Del Kerlin, Assistant Wastewater Treatment Superintendent
West Yost Associates
Advisory Panel

APPROVED: ______________________________
H. Dixon Flynn -- City Manager

SS 09/13/00

City of Lodi – Wastewater Treatment Master Plan
Summary of Major Alternatives

Alternative	Description	Major Components	Additional Land	Comments	Cost of Alternatives (All costs in million $) Capital	Cost of Alternatives Annual O&M	Cost of Facility Improvement (all alts) Capital	Cost of Facility Improvement Annual O&M	Score [1]	Total Annualized [2] Cost
1. DC-D	Discharge to Dredger Cut	• Tertiary Treatment [3] • 500 af Additional Storage (800 af total, 1 month storage) • Source Control	--	• Project based on current permit • No dilution in Dredger Cut • May be unable to meet future requirements.	17.28	0.61	16.52	0.58	26	4.57
2. DC-W	Discharge to Dredger Cut with wetlands for polishing treatment and storage	• Tertiary Treatment [3] • Wetlands • 250 af Additional Storage	• 130 ac west of W.S. Facility @ Bishop Cut	• No dilution in Dredger Cut	18.78	0.64	16.52	0.58	30.5	4.75
3. BC-D	Discharge to Bishop Cut through on outfall pipeline	• Outfall Pipeline (approx. 1.3 miles) • Tertiary Treatment [3] • Source Control	• Easements for pipe	• Better dilution in Bishop Cut	16.66	0.59	16.52	0.58	32.5	4.49
4. BC-W	Discharge to Bishop Cut through an outfall wetlands	• Outfall Wetlands • Tertiary Treatment [3]	• 100 ac west of W.S. Facility @ Bishop Cut	• Better dilution in Bishop Cut	17.18	0.62	16.52	0.58	37.5	4.57
5. BC-PD	Partial discharge to Bishop Cut through an outfall pipeline, partial percolation disposal	• Outfall Pipeline (approx. 1.3 miles) • Source Control • Denitrification • Percolation Basins/Fields	• 260 ac east of Thornton Road • Easements for pipe	• Secondary Treatment [4] • Some crop potential in percolation basins	14.79	0.37	16.52	0.58	34.5	4.08
6. BC-PW	Partial discharge to Bishop Cut through an outfall pipeline, partial percolation disposal	• Outfall Wetlands • Denitrification • Percolation Basins/Fields	• 260 ac east of Thornton Road • 60 ac west of W.S. Facility @ Bishop Cut	• Secondary Treatment [4] • Some crop potential in percolation basins	14.24	0.34	16.52	0.58	37.5	4.00
7. LD	Land discharge – complete effluent disposal through percolation and agricultural irrigation reuse	• Percolation Basins/Fields • Denitrification	• 400 ac east of Thornton Road	• Secondary Treatment [4] • Some crop potential in percolation basins	17.40	0.28	16.52	0.58	36.5	4.25

1. Score is based on various criteria as described in the draft Master Plan. Cost is not a factor in the scoring.
2. Annualized cost is total of annual O&M plus 10% of capital cost based on assumed financing.
3. Tertiary treatment is additional filtration and disinfection. Treated water is then usable for food crop irrigation.
4. Secondary treatment is what is done currently. Treated water for irrigation is limited to use on animal feed crops.

AlternativesSummary

CITY OF LODI

PUBLIC WORKS DEPARTMENT

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September 15, 2000

Robert A. Beggs, P.E.
West Yost & Associates
1260 Lake Blvd., Ste. 240
Davis, CA 95616

Advisory Panel

SUBJECT: Wastewater Treatment Plant Master Plan

We will be discussing the Wastewater Treatment Master Plan at the City Council's next informal Shirtsleeve Session. This meeting will be held Tuesday, September 19, 2000, at 7 a.m. in the City Council Chamber, Carnegie Forum, 305 West Pine Street. I hope you will be able to attend.

We have included all of the options discussed to date (see enclosure), and, at this point, are not requesting the Council decide on an option.

If you have any questions, please call me at (209) 333-6759.

Sincerely,

Richard Prima

Richard C. Prima, Jr. lm
Public Works Director

RCP/lm

Enclosure

cc: City Clerk

NSS

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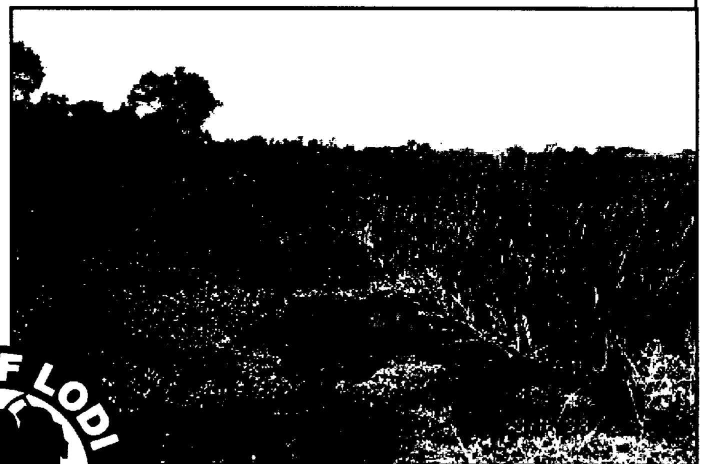
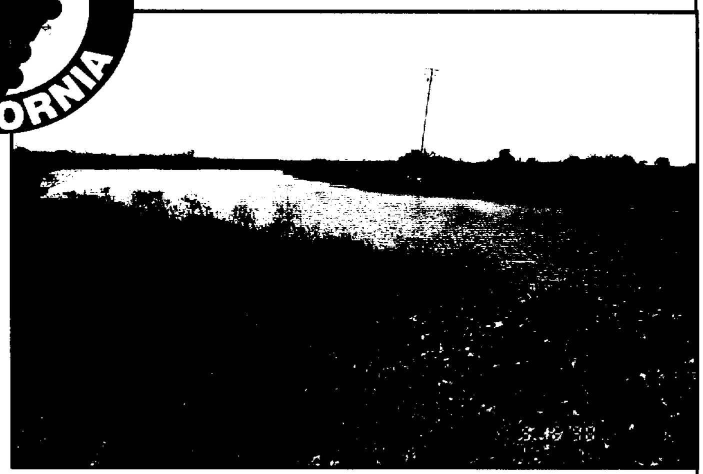
CITY OF LODI

September 2000

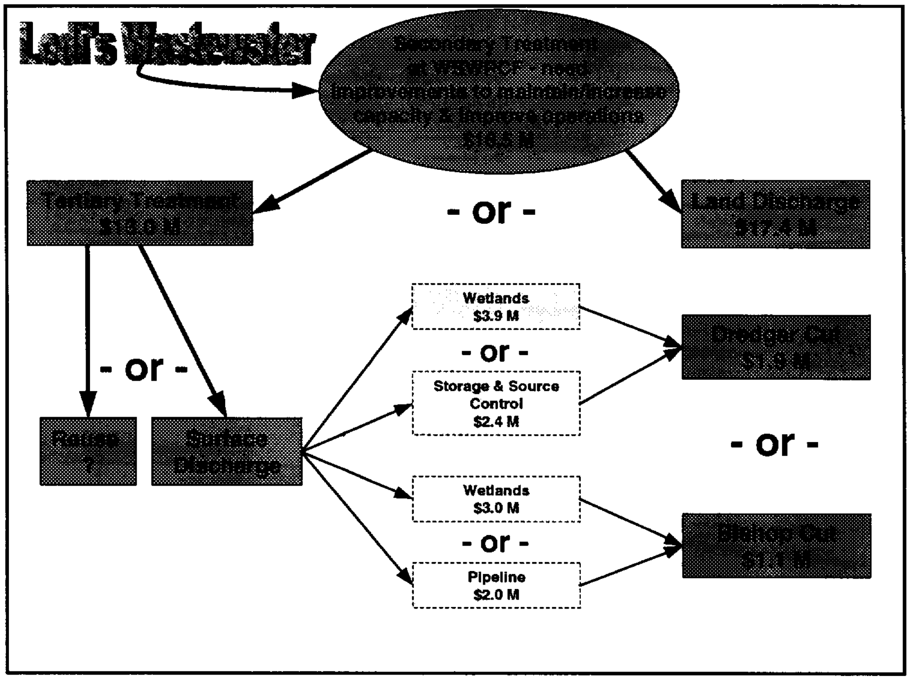




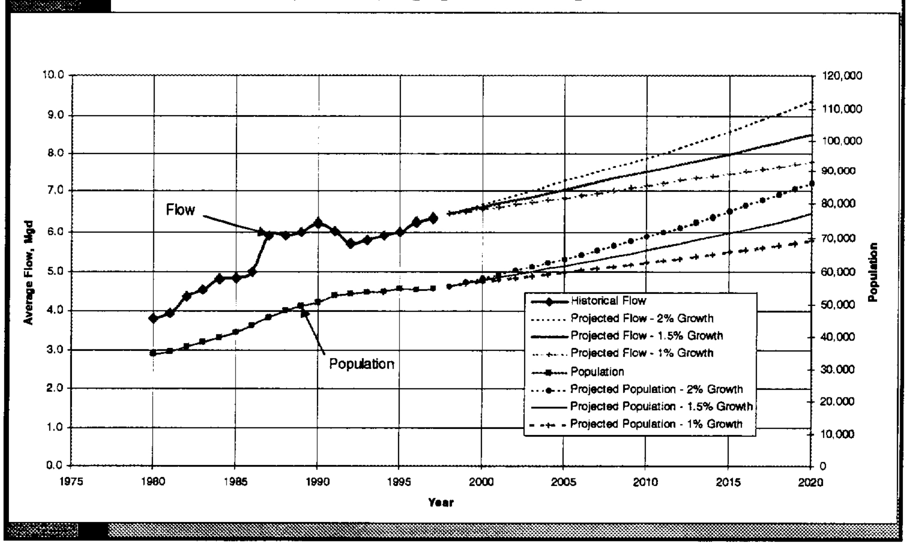
REPORT CORE SECTIONS

WASTEWATER MASTER PLAN

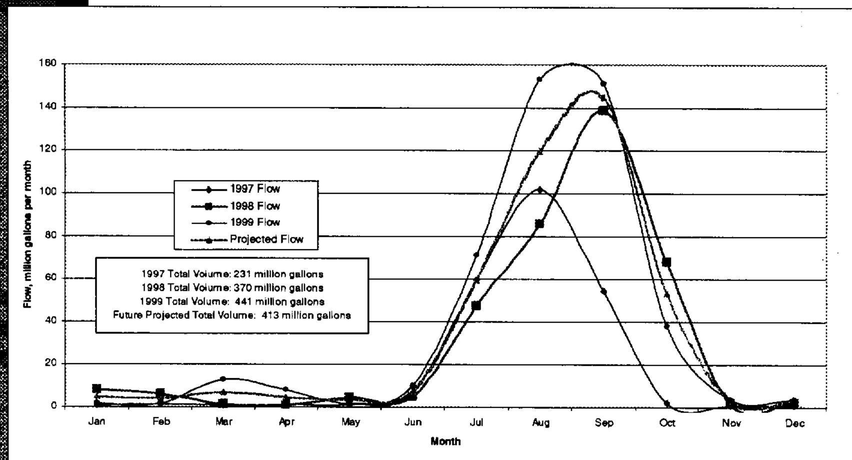
WEST YOST & ASSOCIATES
Consulting Engineers
in association with:
BROWN AND CALDWELL
DJH ENGINEERING



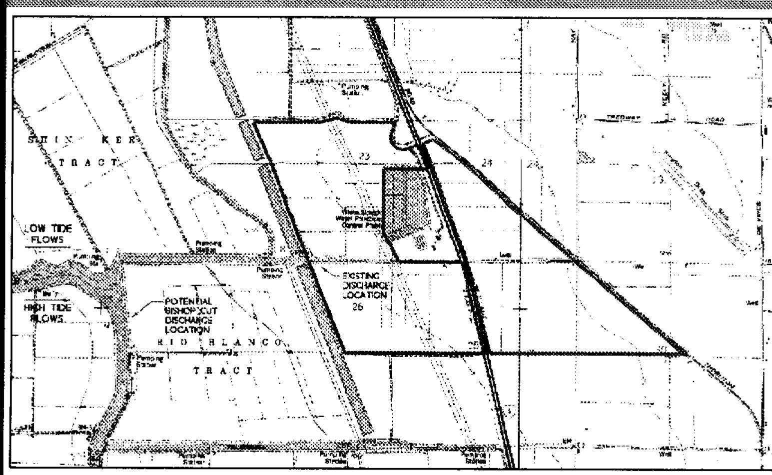
Population and Wastewater Flow



1997-1999 and Projected Industrial Wastewater Flows

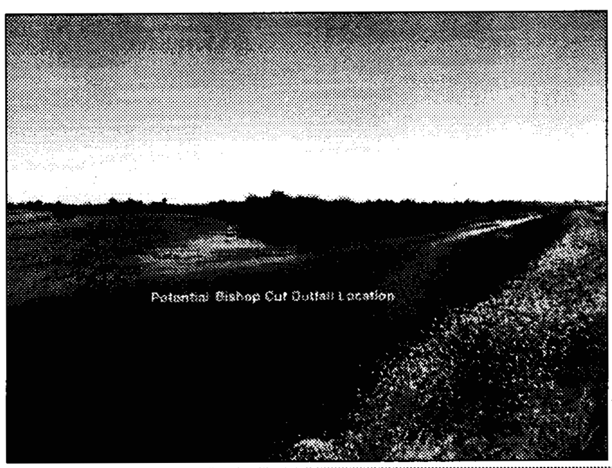


Discharge & Reuse Locations



Dredger Cut Outfall Location

Potential New Outfall Location in Bishop Cut



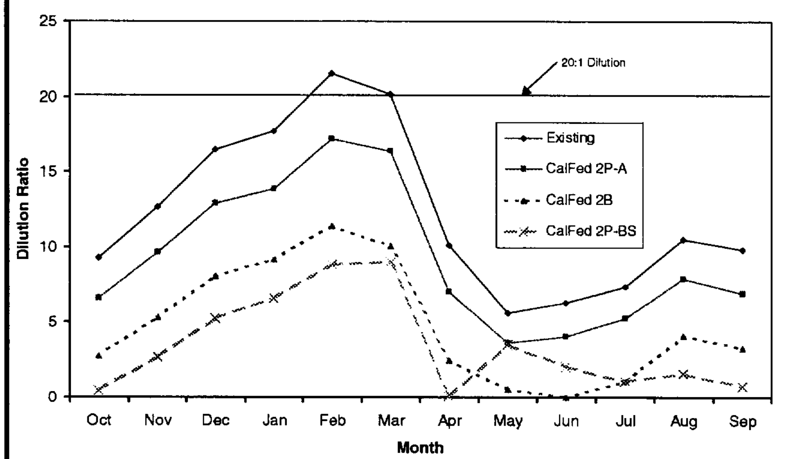
Requirements Comparison

- Discharge Requirements Significantly More Difficult to Meet in Dredger Cut than Bishop Cut
  - Dissolved oxygen - 5 point per 1 liter
  - Temperature -
  - Trace toxins -
  - Ammonia

Surface Discharge Alternatives

- Continuing to Discharge into Dredger Cut
  - Poor dilution
  - Discharge requirements difficult to meet
- Outfall to Bishop Cut
  - Improved dilution (still limited)
  - Easier to meet Waste Discharge Requirements

Bishop Cut Dilution



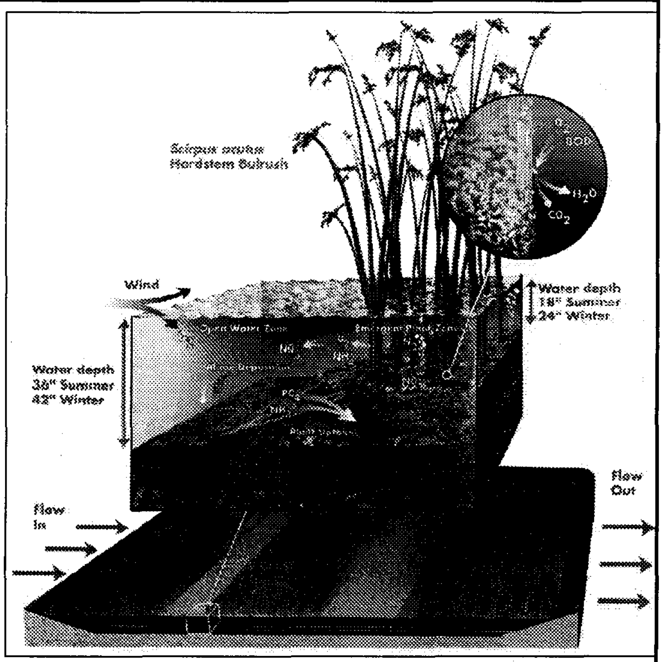
Subjective Evaluation Criteria

Criteria	Relative Weighting
Compliance with Potential Future Discharge Standards	1.5
Reliability	1.5
Flexibility	1.5
Ease of Operation and Maintenance	1.0
Ease of Implementation	1.0
Environmental Impacts	1.0
Safety	1.0
Potential Recreational/Open Space Benefits	0.5
Aesthetics	0.5
Secondary Economic Benefits	0.5
Resource Management Considerations	0.5

Wetlands Performance Characteristics

- Reduces Temperature
- Reduces Nitrates
- Reduces Toxicity
- Provides Storage

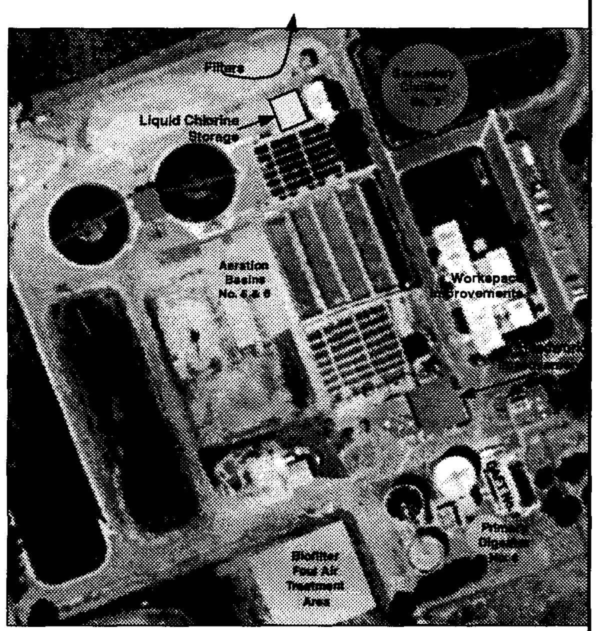
Typical Wetlands



Treatment Plant Design and Operational Criteria

- Treatment Requirements
- Treatment Redundancy
- Staff and Public Safety
- Americans with Disabilities Act
- 100-Year Flood Protection
- Power Backup and Control Redundancy
- Staffing Levels
- Operational Flexibility
- Control and Automation

Aerial View of Master Plan Facilities



Item	Estimated Capital Cost, $	Notes
Provide Additional Capacity		
Modify Domestic Pumps	210,000	Previously Deferred
Replace Industrial Pump, add 3rd	260,000	
WAS Thickening Improvements	1,640,000	Previously Deferred
New Anaerobic Digester	1,710,000	Previously Deferred
Sludge Lagoon Capacity	930,000	Previously Deferred
Subtotal	$4,750,000	
Comply with Regulatory Requirements		
2 New Aeration Basins	4,440,000	Restore previous capacity
New Secondary Clarifier	2,700,000	Restore previous capacity
RAS/WAS Improvements	1,360,000	Restore previous capacity
Liquid Hypochlorite, Bisulfite	200,000	Safety improvements
Storage Pond Aeration	280,000	Odor control
Operations Building Improvements	150,000	ADA
Odor Control Facilities	890,000	
Subtotal	$10,050,000	
Rehabilitation & Operations Improvements		
Headworks Improvements	260,000	Rehabilitation
Influent Screening	660,000	Performance
Septage Receiving Station	60,000	Operations
Clarifier Polymer Feed System	280,000	Redundancy
Control System Upgrades	319,700	Operations
Miscellaneous Operational Upgrades	140,000	Operations
Subtotal	$1,720,000	
Common Improvements Total	$16,520,000	
Value of Previously Deferred Components	$4,390,000	

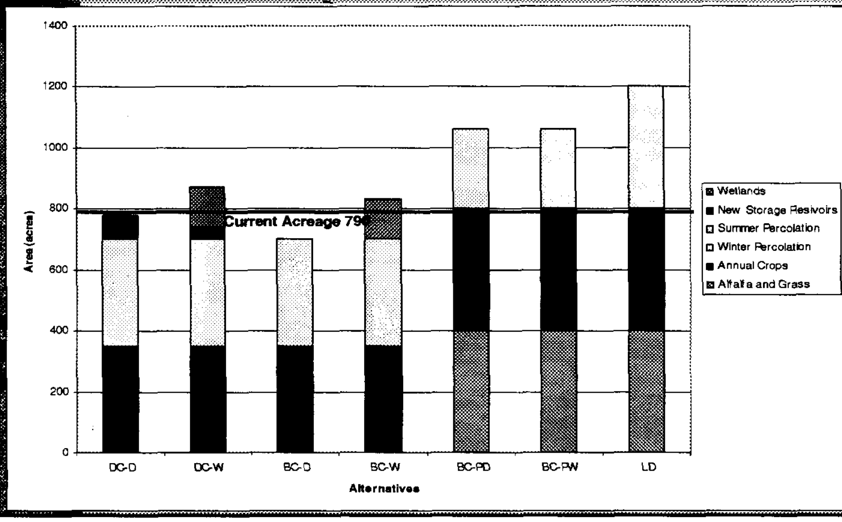
Treatment plant costs associated with individual alternatives not included in this tables

Improvements Common to All Process Alternatives

Alternatives for Effluent Discharge, Reuse & Storage

- Year-Round Surface Discharge
  - DC-D: Direct surface discharge to Dredger Cut
  - DC-W: Wetland discharge to Dredger Cut
  - BC-D: Direct surface discharge to Bishop Cut
  - BC-W: Wetland discharge to Bishop Cut
- Seasonal Surface Discharge
  - BC-PD: Direct seasonal surface discharge to Bishop Cut
  - BC-PW: Wetlands seasonal discharge to Bishop Cut
- LD: Land Discharge

Land Requirements for Proposed Alternatives



Summaries & Conclusions

- City Has Sufficient Land for Industrial Wastewater and Biosolids
- More Land Needed for Municipal Effluent, Especially if Wetlands and Percolation Basins Are Desired
- Land Discharge Would Require Approximately 400 Net Additional Acres of Sandy Land
- Additional Nitrogen Removal Recommended with Percolation Basins
- Groundwater Level Control Measures May Be Needed for Winter Percolation Disposal

Subjective Comparison of Alternatives

Subjective Criteria	Weightings	Alternatives & Relative Scores						
		DC-D	DC-W	BC-D	BC-W	BC-PD	BC-PW	LD
Compliance with Future Discharge Requirements	1.5	1	2	3	4	3	3	5
Reliability	1.5	3	2	4	3	4	4	4
Flexibility	1.5	1	2	3	3	4	4	3
Ease of Operation and Maintenance	1	3	3	3	3	4	4	4
Ease of Implementation	1	4	3	3	3	3	3	2
Environmental Impacts	1	3	4	3	5	3	4	4
Safety	1	4	4	4	4	4	4	5
Open Space/Recreational Benefits	0.5	1	4	1	4	2	3	2
Aesthetics	0.5	3	5	3	5	3	4	2
Secondary Economic Benefits	0.5	1	2	1	2	1	2	1
Resource Management Considerations	0.5	4	4	4	4	2	3	2
Totals		28	35	32	40	33	38	34
Totals (weighted)		26	30.5	32.5	37.5	34.5	37.5	36.5

0 = Worst
5 = Best

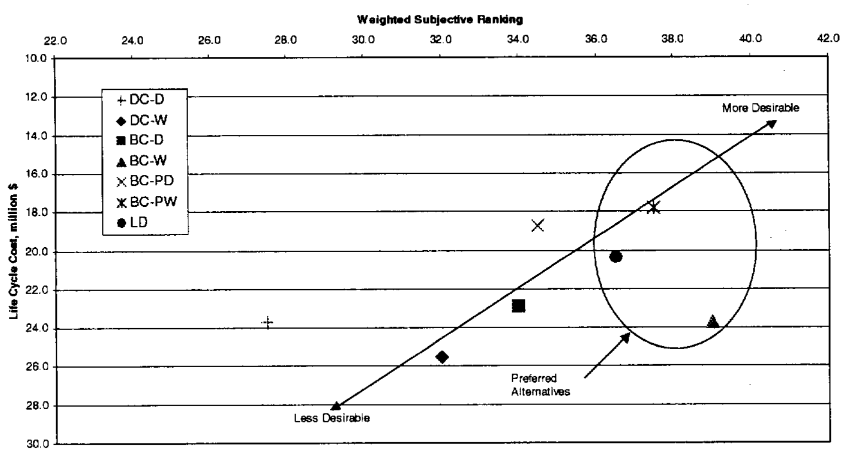
Costs for Major Alternatives

Alternative	Capital Costs (million $)	Annual O&M (thousand $)	Life Cycle (million $)
DC-D*	17.3	610	23.7
DC-W*	18.8	638	25.5
BC-D*	16.7	592	22.9
BC-W*	17.2	618	23.7
BC-PD	14.8	370	18.7
BC-PW	14.2	336	17.8
LD	17.4	280	20.4

* Includes tertiary filtration and disinfection

Capital cost of additional treatment plant improvements not included above which are common to all alternatives: $16.4 million

Comparison of Major Alternatives



Est. Total Capital Costs of the Most Cost Effective Alternatives

Alternative	Capital Costs (million $)		
	Common Elements	Specific Elements	Total
BC-W	16.5	17.3	33.8
BC-PW	16.5	14.2	30.7
LD	16.5	17.4	33.9

Recommendations

- Perform Pilot Testing of Winter Percolation Disposal
- Submit Application to Regional Water Quality Control Board for New Discharge Permit for Both Discharge to Bishop Cut and Winter Percolation
- Begin Discussions about Purchasing or Otherwise Obtaining Operating Control of Land Needed for Alternative Implementation
- Begin Public Participation Process

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SECTION 1. EXECUTIVE SUMMARY

pending

SECTION 2. INTRODUCTION

pending

SECTION 3. FLOW AND LOADING PROJECTIONS

This section quantifies existing wastewater flows and loadings, and presents projections for future flow rates and loadings through Year 2020. Flows affect the hydraulic design and sizing of pumps, pipes, and other system components. Loadings affect the biological treatment process components such as aeration basins and anaerobic digesters. Projections are presented for both domestic and industrial sewer wastewater flows. In addition, this section presents an initial analysis of infiltration and inflow (I/I) into the City's municipal wastewater collection system.

POPULATION PROJECTIONS

The City's most recent General Plan was completed in 1991. The target population through 2007, the end of the General Plan period, was 70,741. This represents a two percent annual growth rate from the 1987 population level of 45,794.

According to the City's 1998 Residential Growth Management Schedule[1], the population of Lodi was 55,681 in January 1998. Population projections for San Joaquin County and its cities have been developed by the San Joaquin Council of Governments for Year 2020. Their projection for Lodi is that the City will grow to a population of 69,156 by 2020 – a growth rate of 0.99 percent. This is the lowest rate of the seven cities in the county. The total county growth rate was estimated to be 1.92 percent. At the General Plan target 2 percent growth rate, the population would be 86,000 by the year 2020. Population projections for 1 percent, 2 percent, and a mid-range value of 1.5 percent through 2020 are shown in Figure 3-1

LAND USE

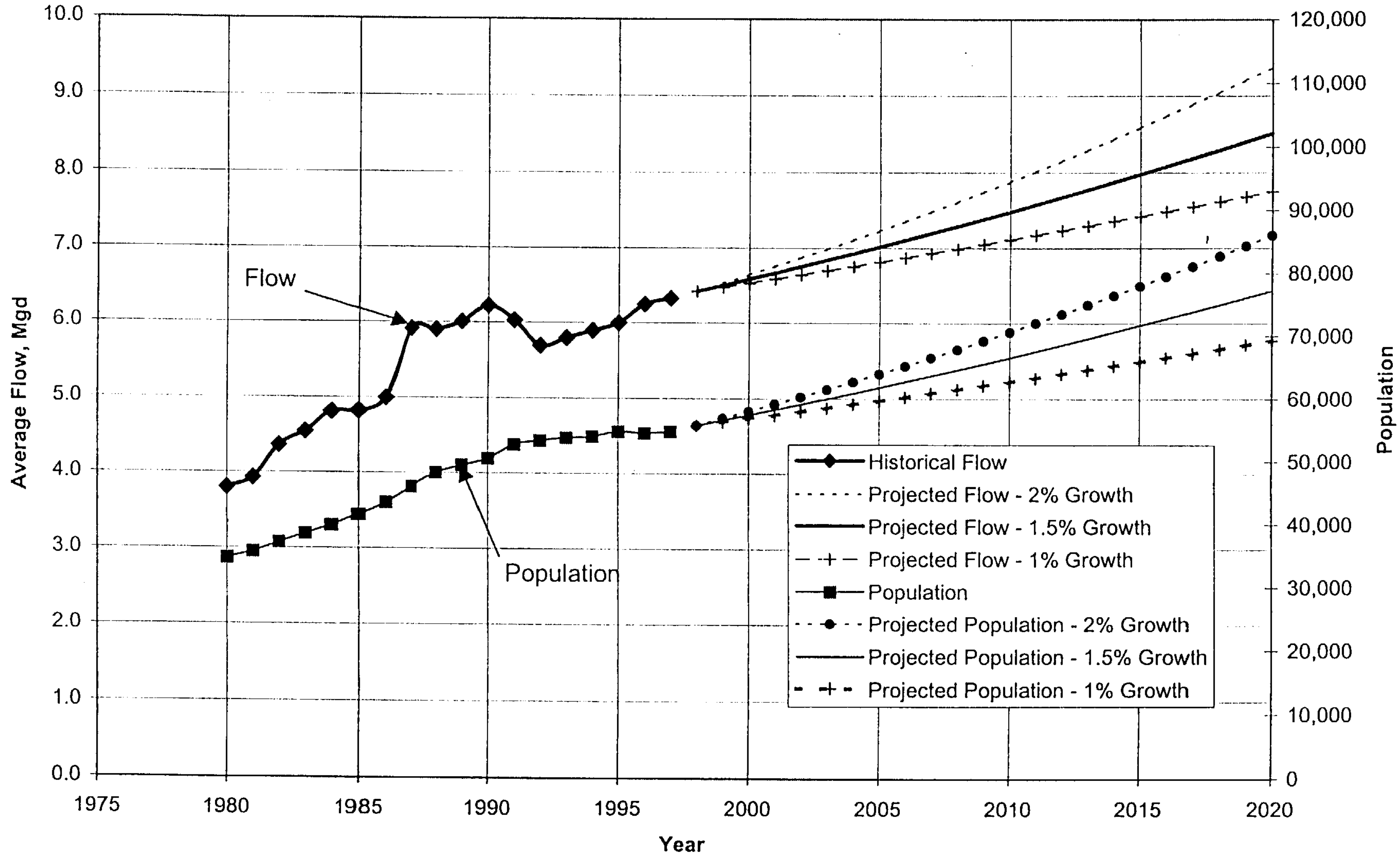
The ratios of future land uses are expected to remain relatively constant over the next 20 years[2]. For residential units, the current proportions are projected to remain approximately constant for at least the next decade at 65 percent single family, 10 percent medium density, and 25 percent high density. If the land uses and residential mix stay constant as expected, wastewater flows should correlate well with projected population.

DOMESTIC WASTEWATER FLOW PROJECTIONS

Average Flow

Historical wastewater flows (annual average) and projected wastewater flows for 1980 through 2020 are shown in Figure 3-1. Flows have generally correlated with population, except for a flow increase during the late 1980s and a flow decrease during the latter stages of the 1987 to 1992 drought. The increase during the late 1980's may be partly explained by calibration problems with the old flow meter around 1985 through 1987. A new flow meter was installed in mid-1988. The decrease in flow during 1991 and 1992 was probably due to water conservation efforts. Since the end of the drought, flows have been increasing slightly faster than population as water conservation efforts have probably lessened. This recent pattern has been evident in wastewater flow data for many municipalities in the area.

FIGURE 3-1. POPULATION AND WASTEWATER FLOW



Based on the historical flows and population for 1980 through 1997, the average wastewater flow per resident was 116 gpd/capita. The wastewater flow rate per resident in 1997 was also 116 gpd/capita. These flows included all commercial customers and some industrial customers. New development in Lodi uses mandated low flow toilets and showerheads. This should reduce average flow per new resident to approximately 97 gpd/capita[3]. Flow projections were developed using the 97 gpd/capita for new growth and 1 percent, 1.5 percent, and 2 percent annual population growth. As can be seen in Figure 3-1, the projected average flow range for 2020 is 7.7 to 9.4 million gallons per day (Mgd). The 1.5 percent growth rate curve (8.5 Mgd at Year 2020) used for planning purposes in this study. Faster growth or higher per capita flows would accelerate the treatment capacity expansion schedule.

Wastewater Flow Peaking Factors

Daily wastewater flows for mid 1994 through early 1999 are shown in Figure 3-2. It is interesting to note that Lodi's wastewater flows are higher in summer months than winter months, which is atypical for cities in the Central Valley. As discussed below, this is probably because Lodi's sewer system has much lower wintertime inflow and infiltration than most other cities' sewer systems. In addition, some of Lodi's businesses have greater activity in the summer months. Because of this pattern, the average annual flow is a better parameter to use for planning purposes than average dry weather flow.

The average annual, peak month, peak day, and peak hour flow rates and peaking factors for the August 1994 through January 1999 period are shown in Table 3-1. These flow rates are based on influent flow meter readings. Seasonal wastewater flow variation is shown in Figure 3-3 along with the maximum monthly flow factors for the period. The daily wastewater flow frequency distribution for this period is shown in Figure 3-4. A graph showing sustained peak flow factors versus number of days is provided as Figure 3-5. The values from Figures 3-3 through 3-5 can be multiplied by projected future average flows for use in sizing treatment and disposal/reuse facilities.

The peak hour flow rate for the period was observed for the storm event peaking on Tuesday, February 3, 1998. The peaking factors shown in Table 3-1 are relatively low compared to most municipal wastewater systems.

Table 3-1. Peak Flow Rates and Peaking Factors

	Flow, Mgd	Peaking Factor
Annual Average	6.2	1.0
Peak Month	7.0	1.13
Peak Day, dry[(a)]	7.3	1.18
Peak Day, wet[(b)]	8.0	1.29
Peak Hour, dry	10.5	1.70
Peak Hour, wet	11.9	1.92

(a) Daily rainfall less than 0.3 inches
(b) Daily rainfall greater than 1.0 inches

FIGURE 3-2. DAILY INFLUENT FLOWS

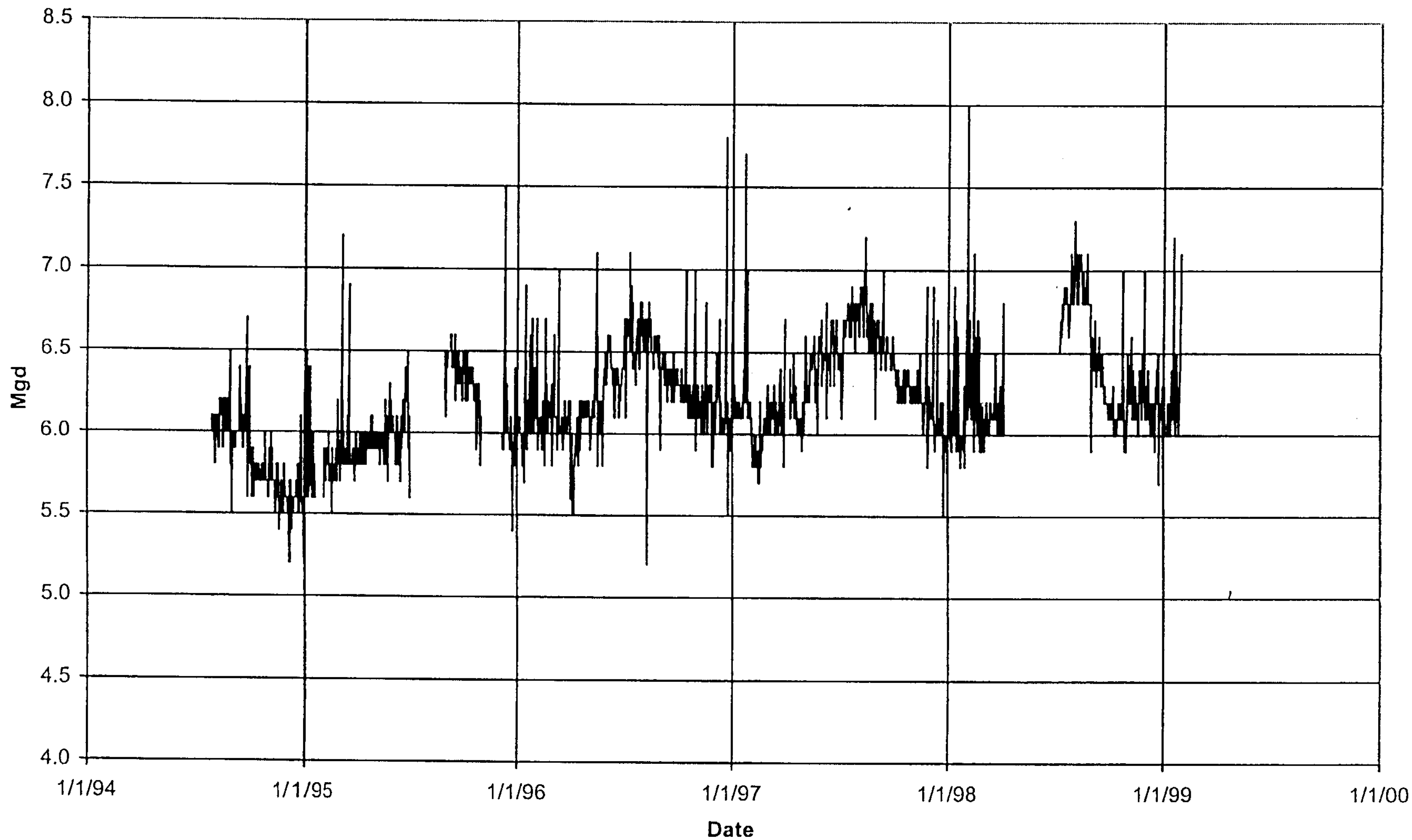


FIGURE 3-3. WASTEWATER FLOW FACTORS BY MONTH (8/94 - 1/99)

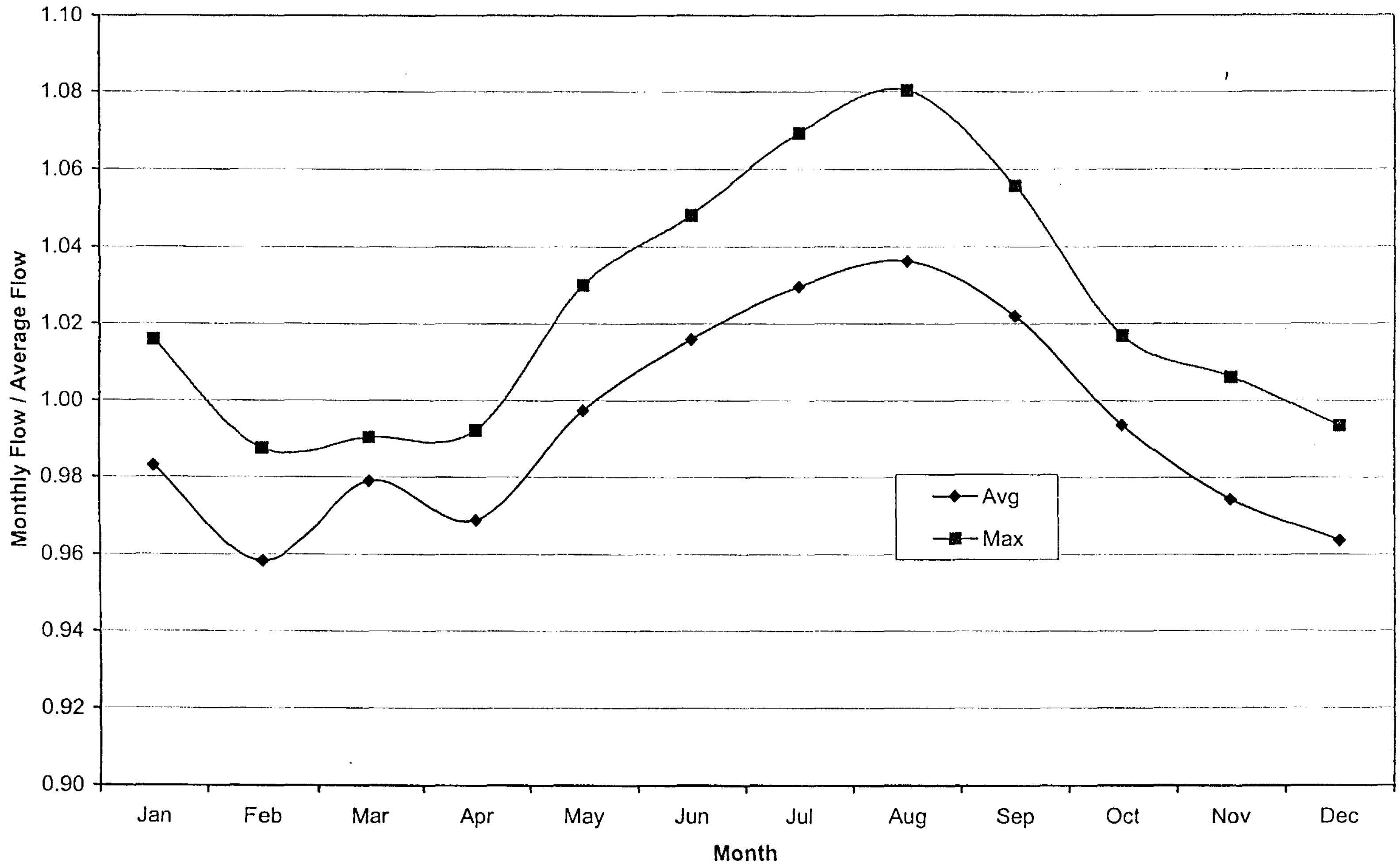


FIGURE 3-4. NORMALIZED DAILY FLOWS FREQUENCY DISTRIBUTION

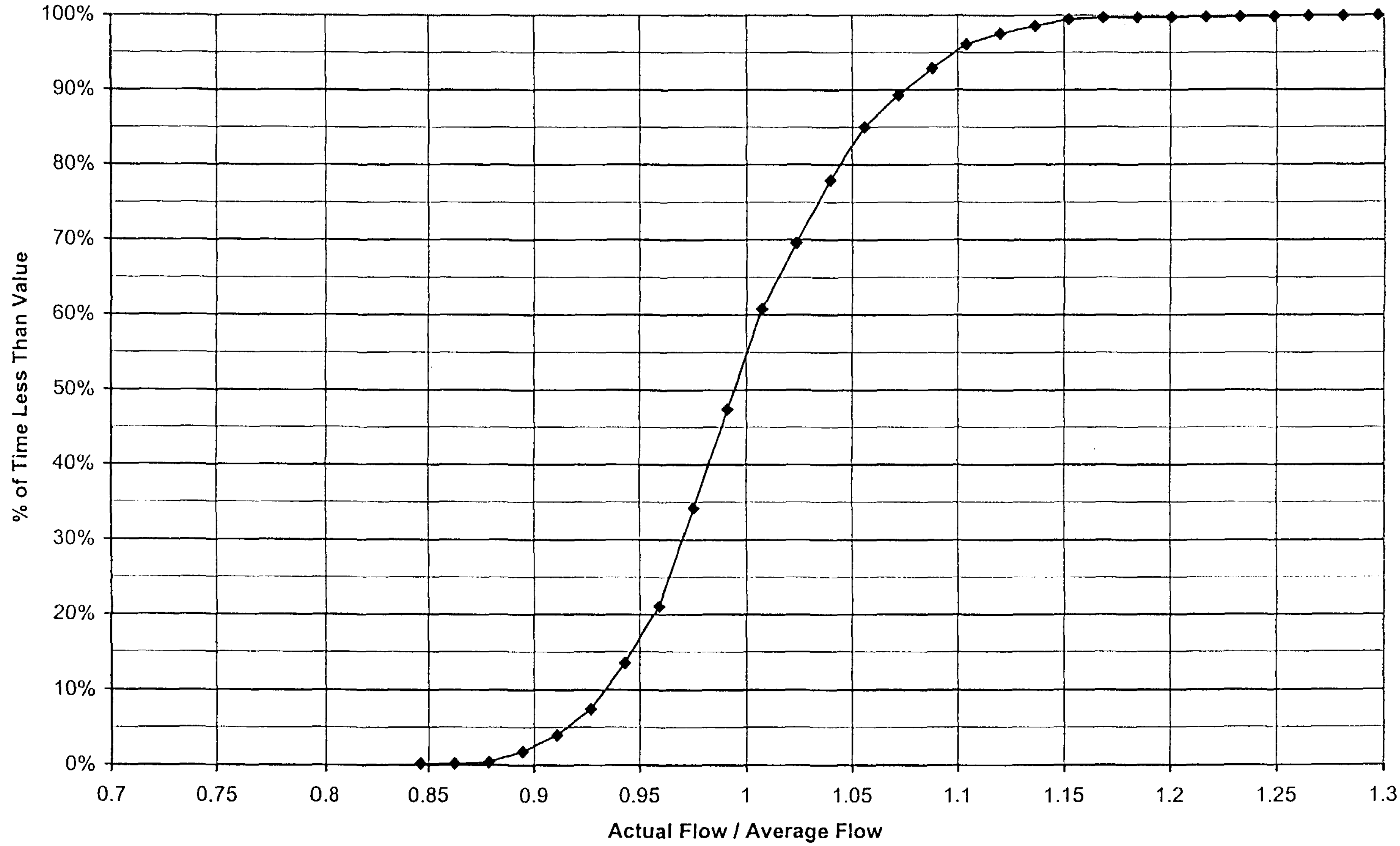
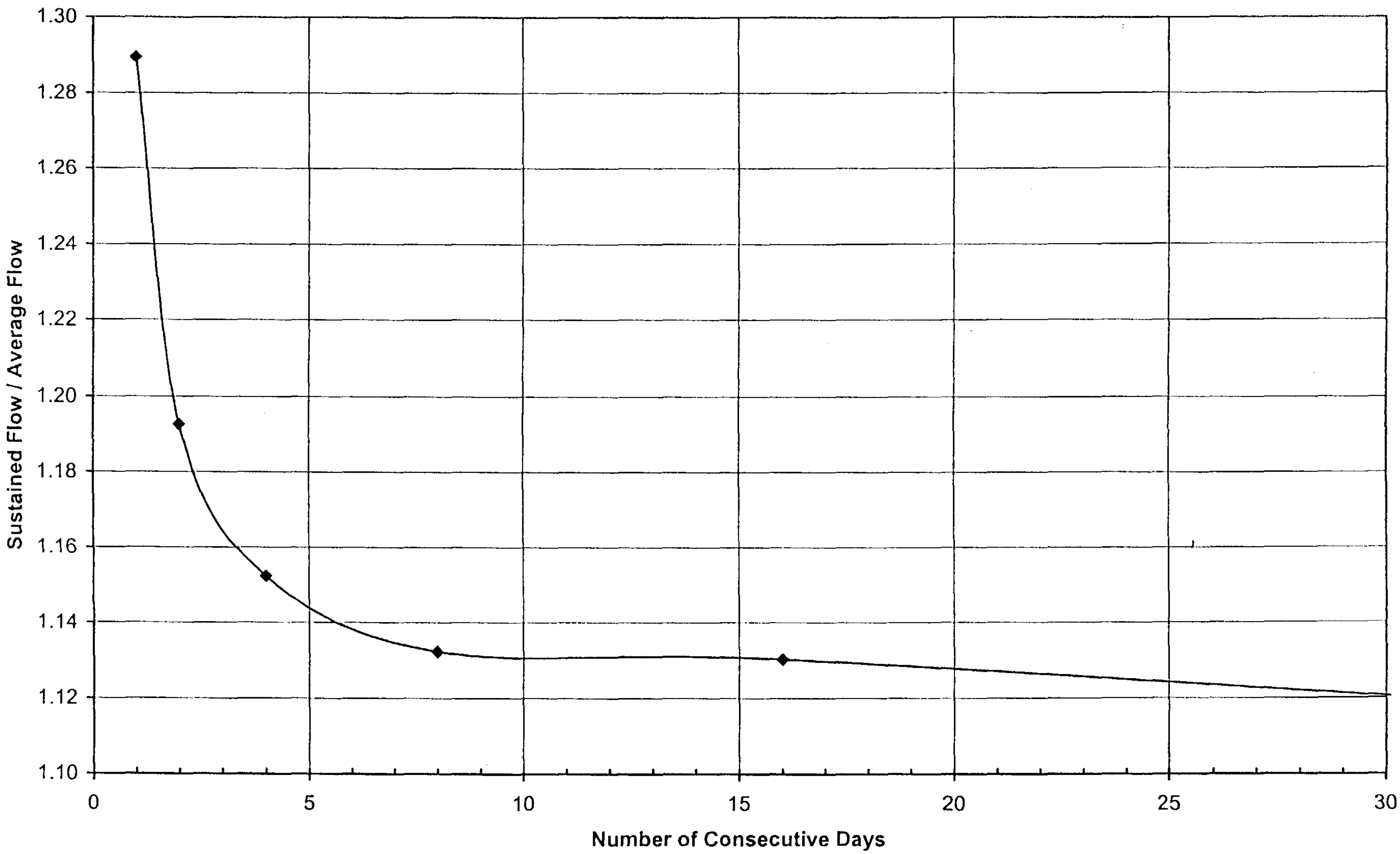


FIGURE 3-5. SUSTAINED PEAK FLOW FACTORS (8/94 - 1/99)



Analysis of Inflow/Infiltration

Direct inflow into wastewater collection systems is defined as surface flows into collection system structures, such as manhole lids. Infiltration is defined as groundwater entering the sewer system through joints and cracks in the system. The purpose of analyzing I/I is to determine whether there is excessive I/I that would be more effective to eliminate through collection system improvements rather than be included in treatment capacity planning.

Groundwater levels are typically highest in late winter months at approximately 40 feet below ground surface. Based on the fact that the wastewater influent flows to the treatment plant are higher in the summer than the winter months (see Figure 3-2), there is no distinguishable infiltration into the Lodi wastewater collection system.

During peak storm events, influent wastewater flows have increased. The average, maximum, and minimum flows during days with rainfall greater than 1.0 inches are compared with the average, maximum, and minimum flows for days with less than 0.3 inches of rainfall in Table 3-2. The peak storm event of February 3, 1998 had an inflow of approximately 2 million gallons over a 24-hour period. The amounts of inflow are very low compared to most wastewater collection systems in the Central Valley of California, and would definitely not be considered excessive.

Table 3-2. Average Inflows During Storm Events (Averages for 1994 through 1998)

	Influent Flow During Dry Periods, Mgd	Influent Flow During Rainstorms,[a] Mgd	Calculated Inflow, gallons
Average for 24 Hours	6.19	6.69	500,000
Average Maximum Hour	7.75	8.96	50,000
Average Minimum Hour	2.94	2.99	2,000

[a] Rainfall greater than 1.0 inch per day

Projected Flows

The average and peak projected flows for planning purposes are listed in Table 3-3. These were calculated using the projected average flows at a 1.5 percent growth rate (Figure 3-1) and the peaking factors from Table 3-1. The frequency distribution and sustained peak flow factors can be used to develop other peaking factors specific to some of the treatment processes.

Table 3-3. Projected Flows, Mgd

	2010	2020
Average	7.5	8.5
Peak Month	8.5	9.6
Peak Day	9.7	11.0
Peak Hour	14.4	16.3
Peak Day, dry weather	8.9	10.0
Peak Hour, dry weather	12.7	14.5

DOMESTIC WASTEWATER QUALITY AND LOADING PROJECTIONS

Concentrations of Major Constituents

The concentrations of major constituents for wastewater entering the Lodi Water Pollution Control Plant are fairly typical of medium strength municipal wastewater. Average and projected concentrations for the major constituents are shown in Table 3-4. Concentrations of minor constituents are addressed in Section 4, Waste Discharge Requirements.

Table 3-4. Average Influent Concentrations of Major Constituents (1995 through 1998)

Item	Units	Historical Average	Projected Year 2010	Projected Year 2020	Existing Treatment Plan Design Criteria
Chemical Oxygen Demand (COD)	mg/L	537	554	565	N/A
Biochemical Oxygen Demand (BOD)	mg/L	265	274	279	220
Total Suspended Solids (TSS)	mg/L	235	243	248	240
Ammonia	mg/L	17.3	17.9	18.2	—
Total Kjeldahl Nitrogen	mg/L	28.5	29.4	30.0	—

Although the land uses and the mix of residential units are not expected to change significantly through Year 2020, new development should have a lower average flow rate per capita. This will result in an increase in the concentrations of major constituents for new development because the constituent loading rates per capita should remain essentially unchanged. This explains the slight increase in concentrations projected over time shown in Table 3-4.

Loading Rates for Major Constituents

Influent loading rates of BOD and TSS have been evaluated for 1994 through 1998. The daily BOD loading rate frequency distribution and sustained peak loading factors are shown in Figures 3-6 and 3-7, respectively. The daily TSS loading rate frequency distribution and sustained peak loading factors are shown in Figures 3-8 and 3-9, respectively. The projected loading rates of major constituents are shown in Table 3-5.

Table 3-5. Projected Average and Sustained Peak Loading Rates in lbs/day

	2010		2020	
Constituent	Average	Sustained Peak 30-Day Loading	Average	Sustained Peak 30-Day Loading
BOD	17,100	19,700	19,800	22,800
TSS	15,200	19,300	17,600	22,400

FIGURE 3-6. NORMALIZED DAILY BOD LOADING FREQUENCY DISTRIBUTION

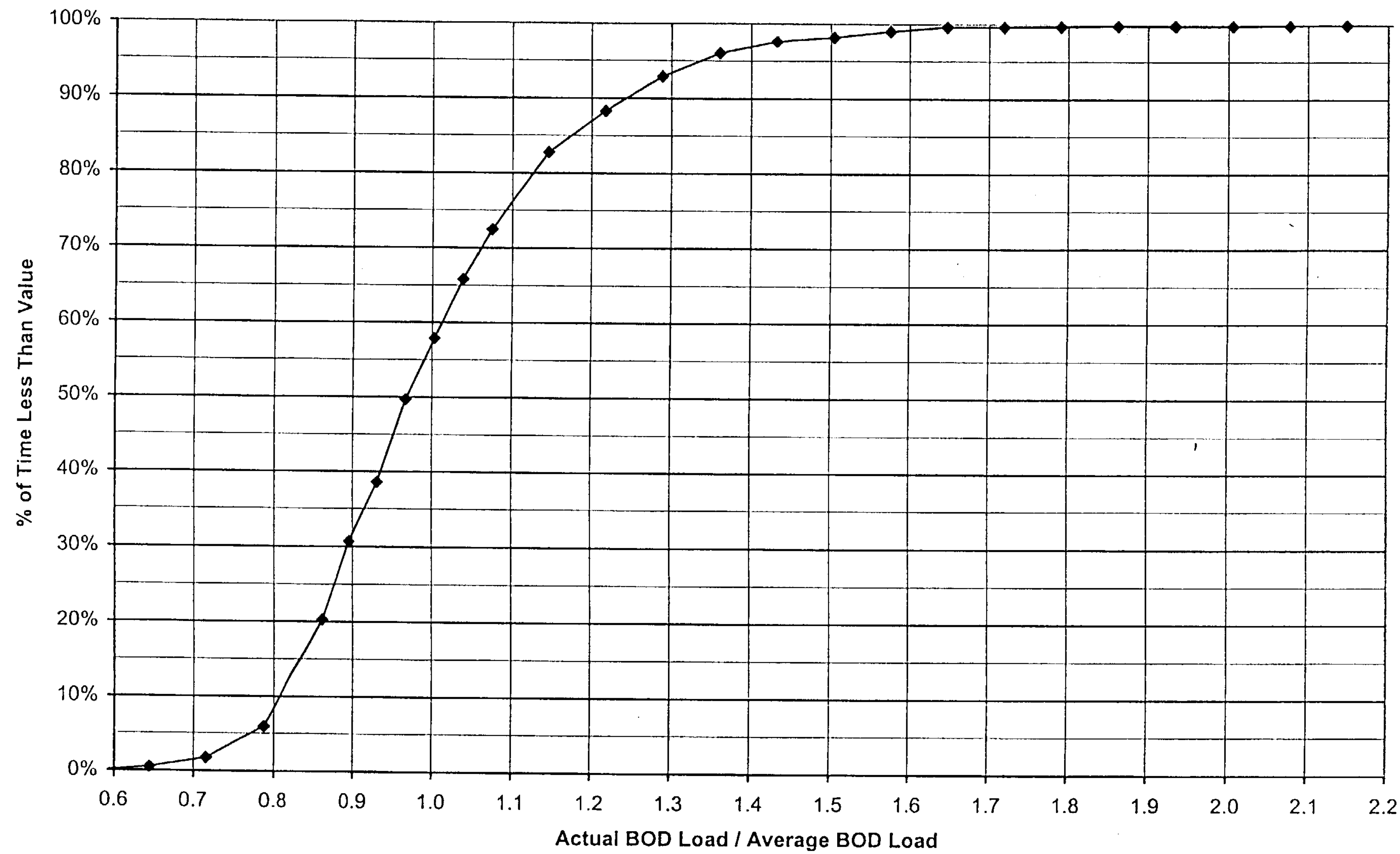


FIGURE 3-7. SUSTAINED PEAK BOD LOADING FACTORS (8/94 - 1/99)

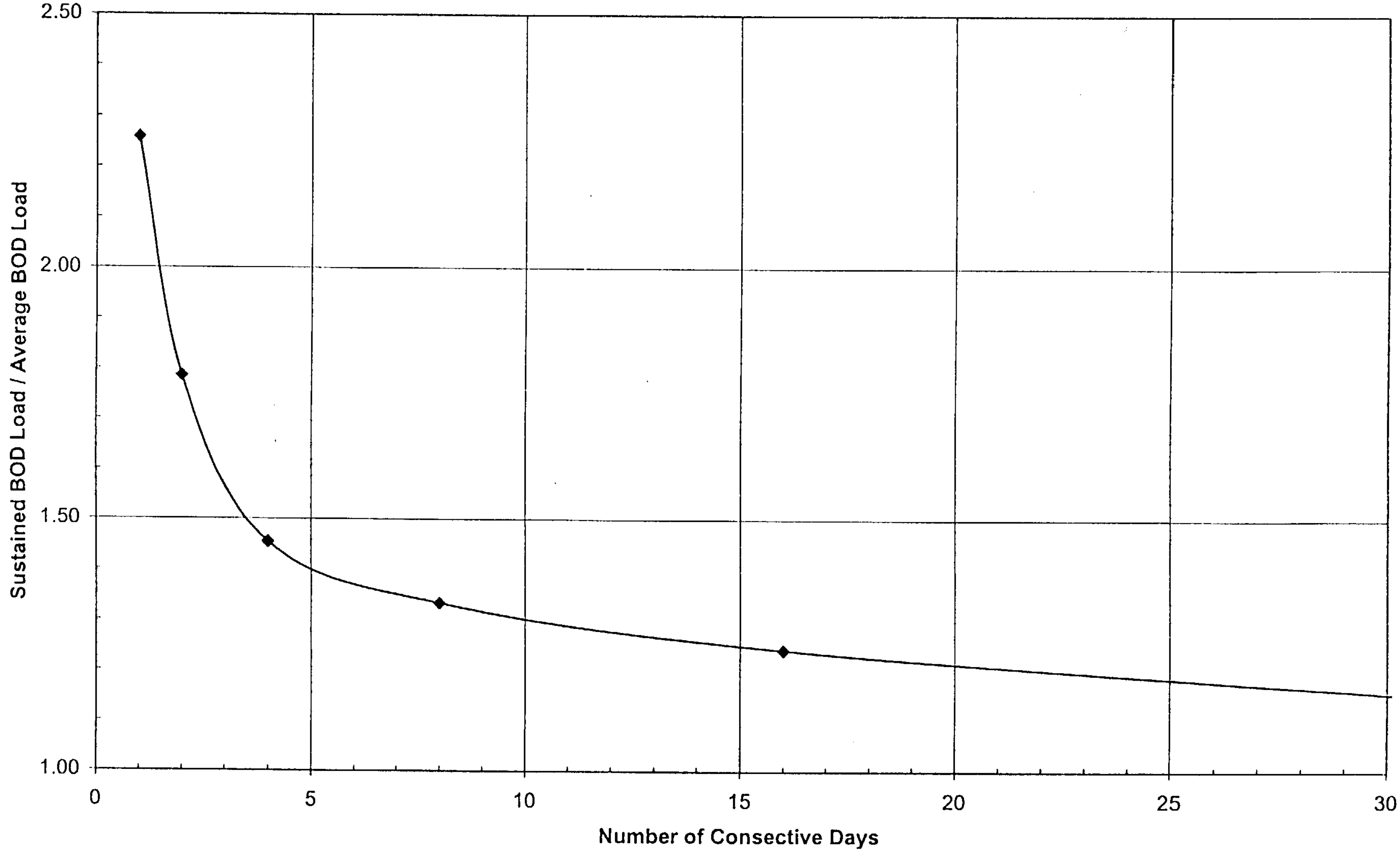


FIGURE 3-8. NORMALIZED DAILY TSS LOADING FREQUENCY DISTRIBUTION

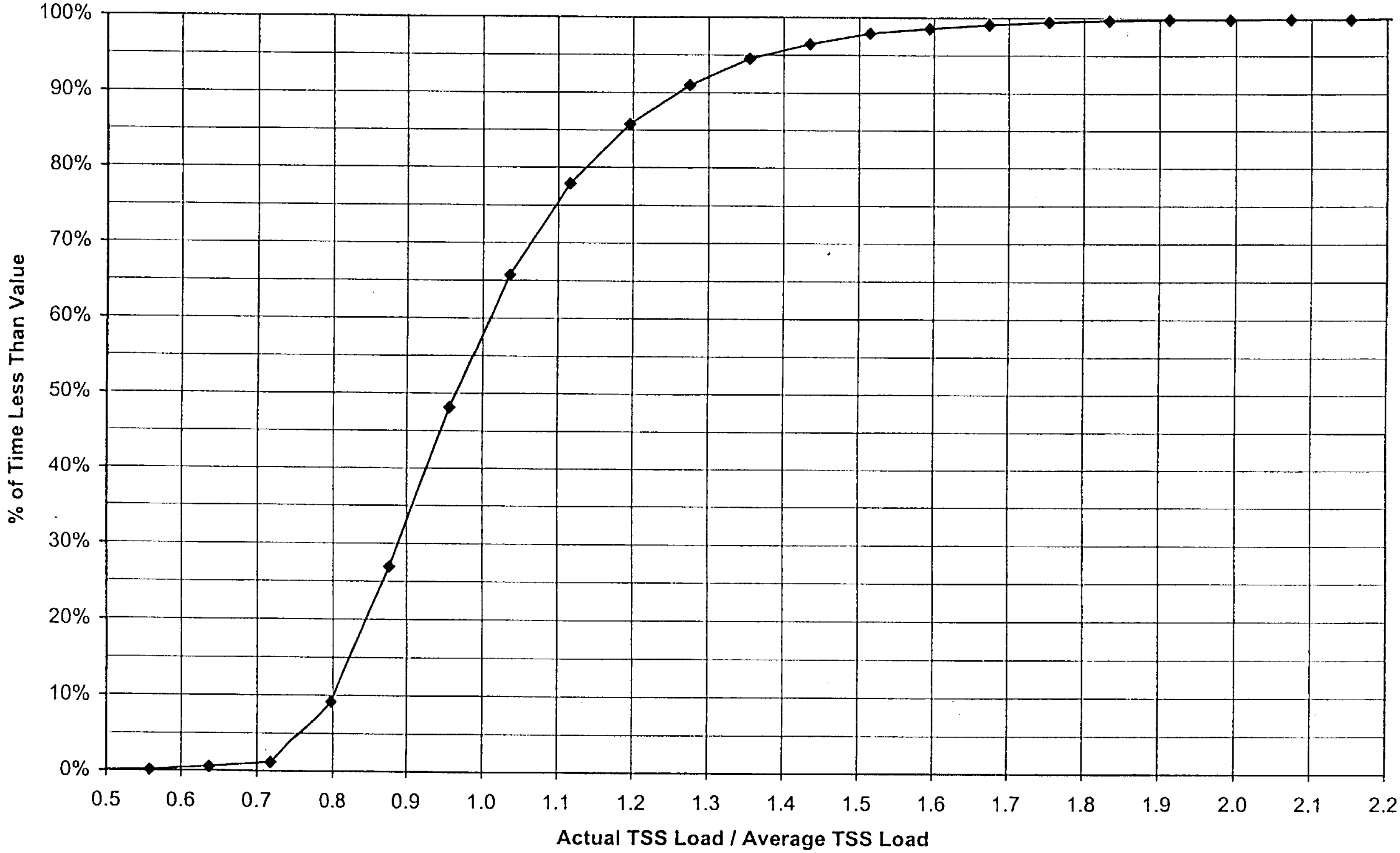
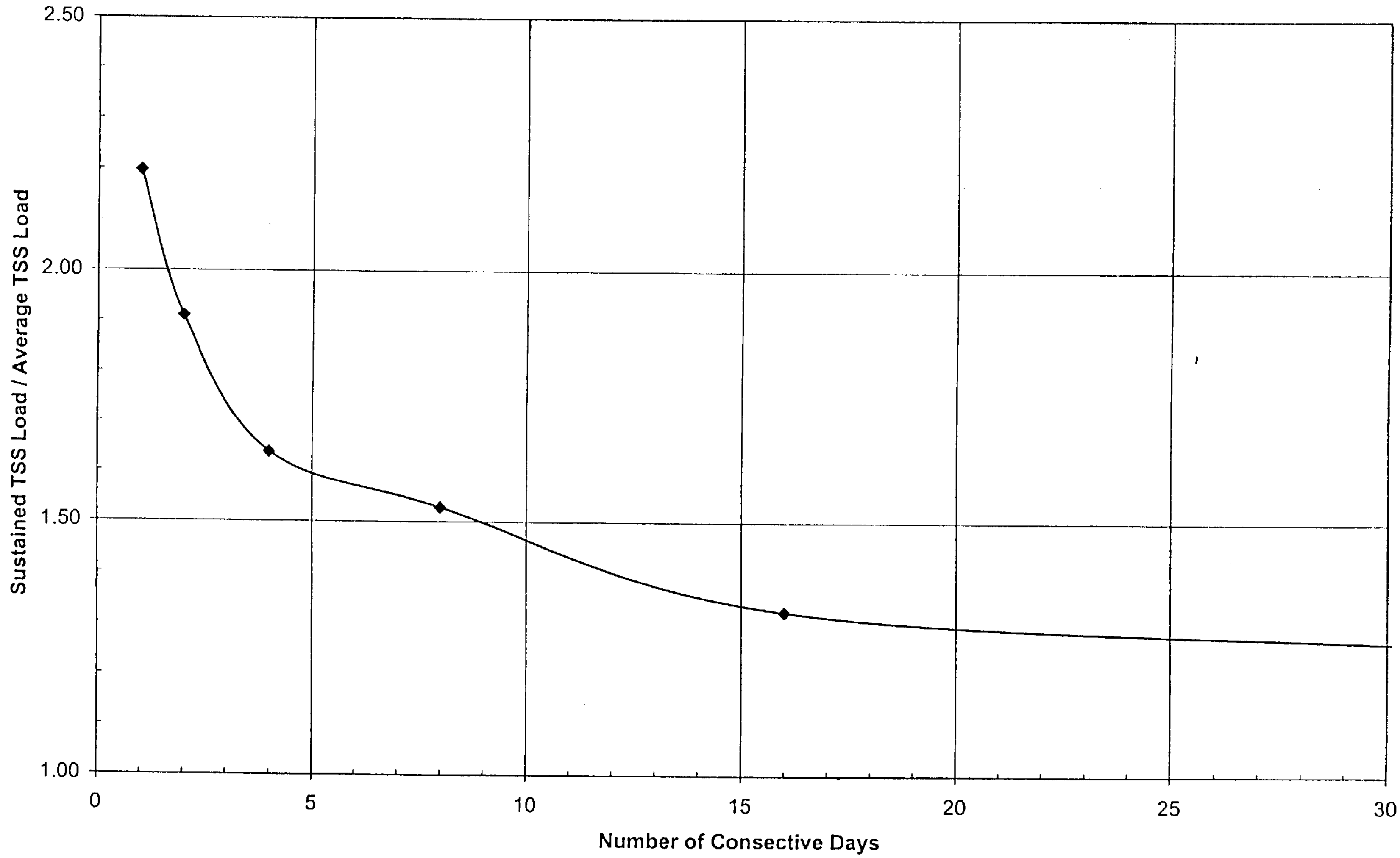


FIGURE 3-9. SUSTAINED PEAK TSS LOADING FACTORS (8/94 - 1/99)



INDUSTRIAL WASTEWATER FLOW AND LOADING PROJECTIONS

The City has a separate 33-inch sewer trunk line which serves the Pacific Coast Producers (PCP) cannery and several small industries. PCP processes primarily apricots during June, and tomatoes and peaches during June through October. PCP also produces sauces and processes other products, but the flows and loads from these operations are very minor.

The smaller industries connected to the industrial sewer system include a cherry packer, metal finishers and several other industries. The combined annual total flow from these industries (other than PCP) is only approximately 14 million gallons versus the 300 to 440 million gallons annually from PCP. The industry names, types, and annual flows are listed in Table 3-6.

Table 3-6. Industries Contributing to the Lodi WWTP Industrial Stream

Industry Name	Industry Type	Discharge, MGY[a]	Discharge Areas within Plant
Interlake	Metal finisher	0.12	Process wastewater
Lodi Iron Works	Iron-casting plant	0.4	Compressor cooling water
M & R Packing	Fruit packing	2.8	Cooling water, fruit wash
Pacific Coast Producers	Cannery	300 – 440	Fruit wash, boiler blow down, caustic peeling of fruit, factory washdown
Pacific Coast Producers	Can-making plant	0.25	Compressor cooling water
RM Holtz Inc.	Industrial Rubber Products	3.3	Process wastewater, cooling water, autoclave blowdown
Valley Industries	Metal finisher	11	Process wastewater

[a] Million gallons per year.

Monthly industrial wastewater flows for 1997, 1998, and a portion of 1999 are shown in Figure 3-10. The 1998 and 1999 flows were much higher than 1997 flows. New equipment is being installed to reduce dilution water flows at PCP. PCP production may expand slightly in the future, but no new major production lines are planned. Based on discussions with PCP management and City staff, the projected flows and loadings were projected using averages for 1998 and 1999 values. Projected flows are shown in Table 3-7 and Figure 3-10. Future peak industrial sewer flow is projected to equal the August 26, 1999 peak of 9.7 mgd. This flow includes tailwater return. Projected loadings are shown in Table 3-7.

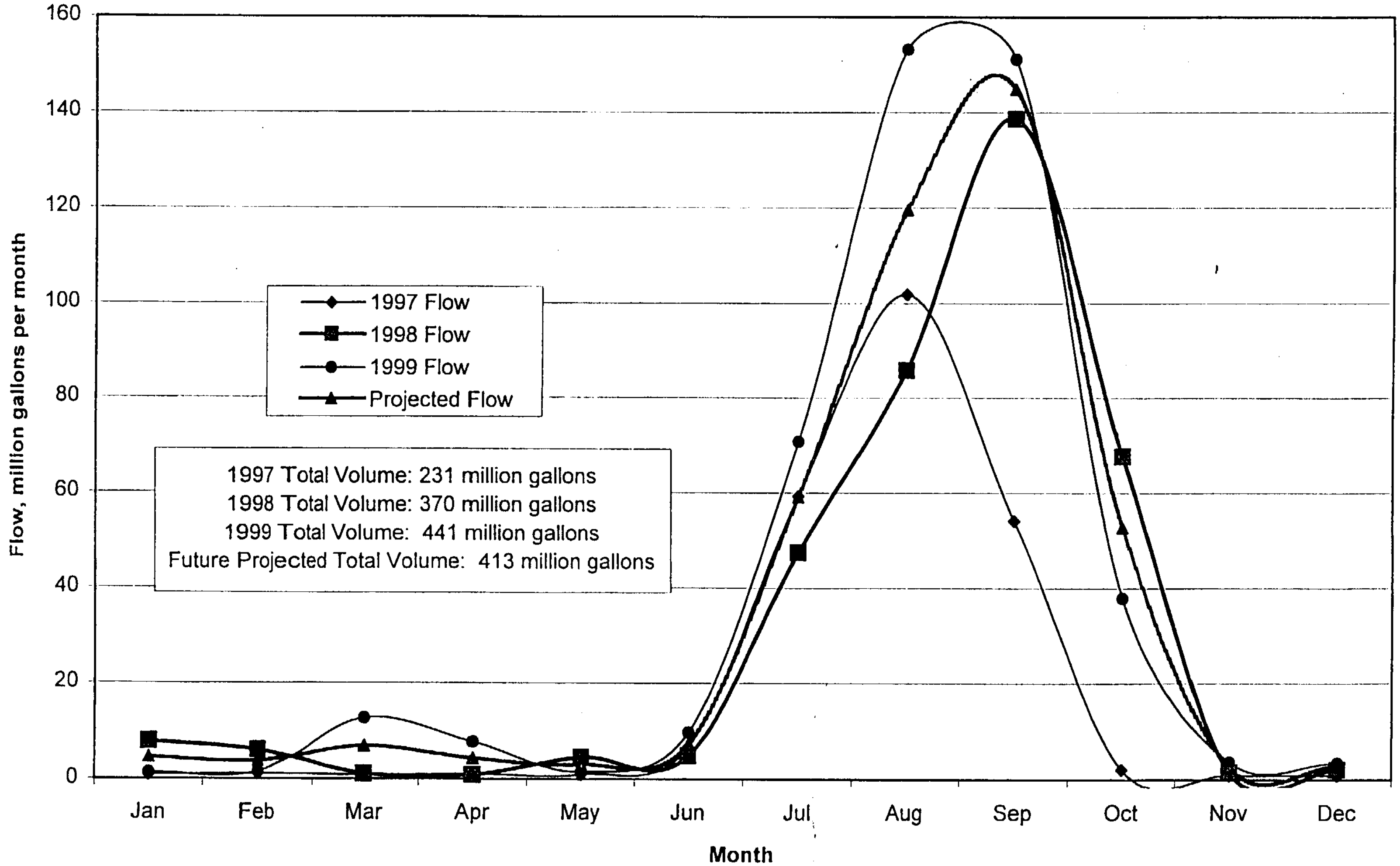
Table 3-7. Projected Industrial Flows and Loadings

Month	Flow, Mgal	BOD, lbs	BOD, mg/L	TSS, lbs	TSS, mg/L
Jan	4.69	10,264	263	10,264	263
Feb	3.89	8,278	255	8,278	255
Mar	6.96	30,355	523	15,964	275
Apr	4.37	9,480	260	9,480	260
May	3.08	6,237	243	6,237	243
Jun	7.37	20,065	326	6,106	99
Jul	59.22	339,631	688	98,637	200
Aug	119.58	2,020,425	2,026	888,599	891
Sep	144.97	2,707,717	2,239	1,321,841	1,093
Oct	52.90	487,436	1,105	271,469	615
Nov	2.69	5,265	235	5,265	235
Dec	2.84	5,644	238	5,644	238
Totals	412.56	5,650,797	N/A	2,647,784	N/A

Note:

PCP flows for Nov through May not sampled – 300 mg/L BOD and TSS assumed.
BOD and TSS for other industries assumed to be an average 100 mg/L.

FIGURE 3-10. 1997 - 1999 AND PROJECTED INDUSTRIAL WASTEWATER FLOWS



REFERENCE

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[1] *City of Lodi Residential Growth Management Schedule 1998*, adopted in accordance with Ordinance #1521 dated September 18, 1991.

[2] Personal phone conversation with Konradt Bartlam, March 1999.

[3] Wastewater flow reduction values calculated from *Wastewater Engineering, Treatment, Disposal, and Reuse*. Tchobanoglous, G. and F.L. Burton. Metcalf and Eddy, Inc. Third Edition. 1991.

SECTION 4. ANTICIPATED DISCHARGE REQUIREMENTS AND ISSUES

INTRODUCTION

The prime objective for the City of Lodi's (City) wastewater facilities is to reliably meet discharge requirements. The purpose of this task was to formulate a set of anticipated and potential future discharge requirements for use in the development and evaluation of upgrades to the City's treatment, reuse, and discharge facilities.

BACKGROUND

Current Processes and Operations

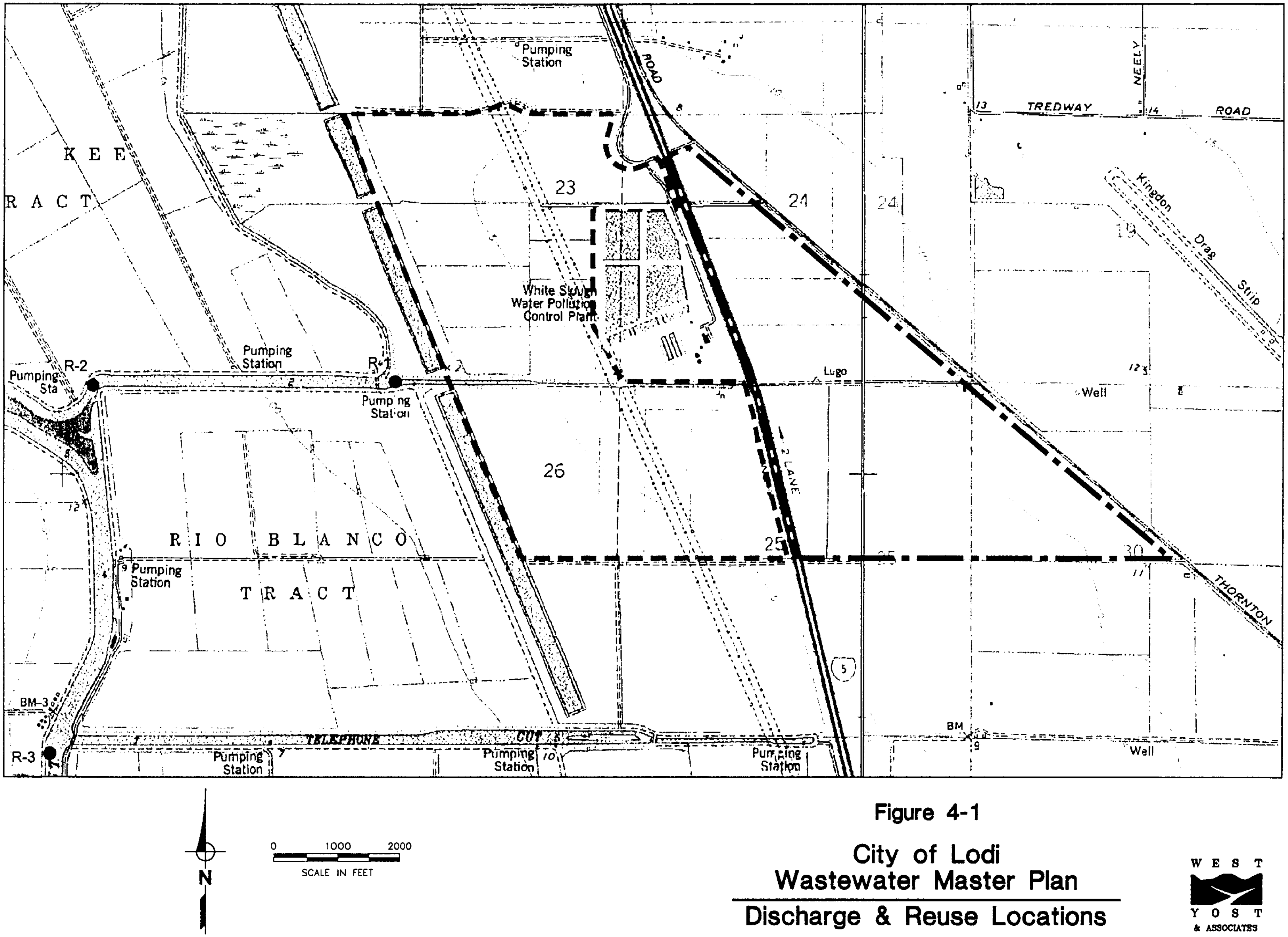
The current treatment process includes primary clarification followed by conventional activated sludge secondary treatment and chlorine gas disinfection. Primary and secondary solids are further treated in anaerobic digesters and a biosolids lagoon. Most treated effluent is either discharged to surface waters or used for agricultural irrigation of animal feed crops. Small amounts of treated effluent are used for the Mosquito Abatement District fish ponds and the NCPA Power Plant. Biosolids are mixed with effluent and land applied on City owned property.

Receiving Waters

The City of Lodi discharges to Dredger Cut, which connects with White Slough and Bishop Cut in the Delta as shown in Figure 4-1. Dredger Cut is a manmade channel which was constructed in the early 1900s to provide drainage for agricultural lands in the area. Dredger Cut, White Slough, Bishop Cut, and other Delta channels are normally dominated by tidal flows. Water from Bishop Cut typically flows to the San Joaquin River and Stockton Deepwater Ship Channel through Disappointment Slough[1] as shown in Figure 4-2. During periods of no exports from the Delta, there is a net flow west from Disappointment Slough towards San Francisco Bay. During periods of high water exports from the Delta, there is a reverse net flow up the San Joaquin River to the confluence with Turner Cut.

Current Discharge Requirements for Municipal Wastewater

Lodi's current (adopted January 2000) discharge requirements for municipal effluent are applied at the discharge into a side slough of Dredger Cut (R-1). The current interim discharge requirements include secondary treatment and disinfection limits, biotoxicity requirements, dissolved oxygen limits, nitrogen loading limits for land application, and related requirements. The most significant current interim discharge requirements related to treatment facility capacities and operation for municipal effluent are listed in Table 4-1. The interim requirements are set to expire in April 2004. The new requirements effective in 2004 are discussed later in the "Future Discharge Requirements" subsection.



CAD FILE: I/213/99-01/FIG-8-2.dwg CFG FILE: WYA2500.PCP_MRG DATE: 02-25-00 10:48am

Figure 4-1

City of Lodi
Wastewater Master Plan

Discharge & Reuse Locations

WEST YOST & ASSOCIATES

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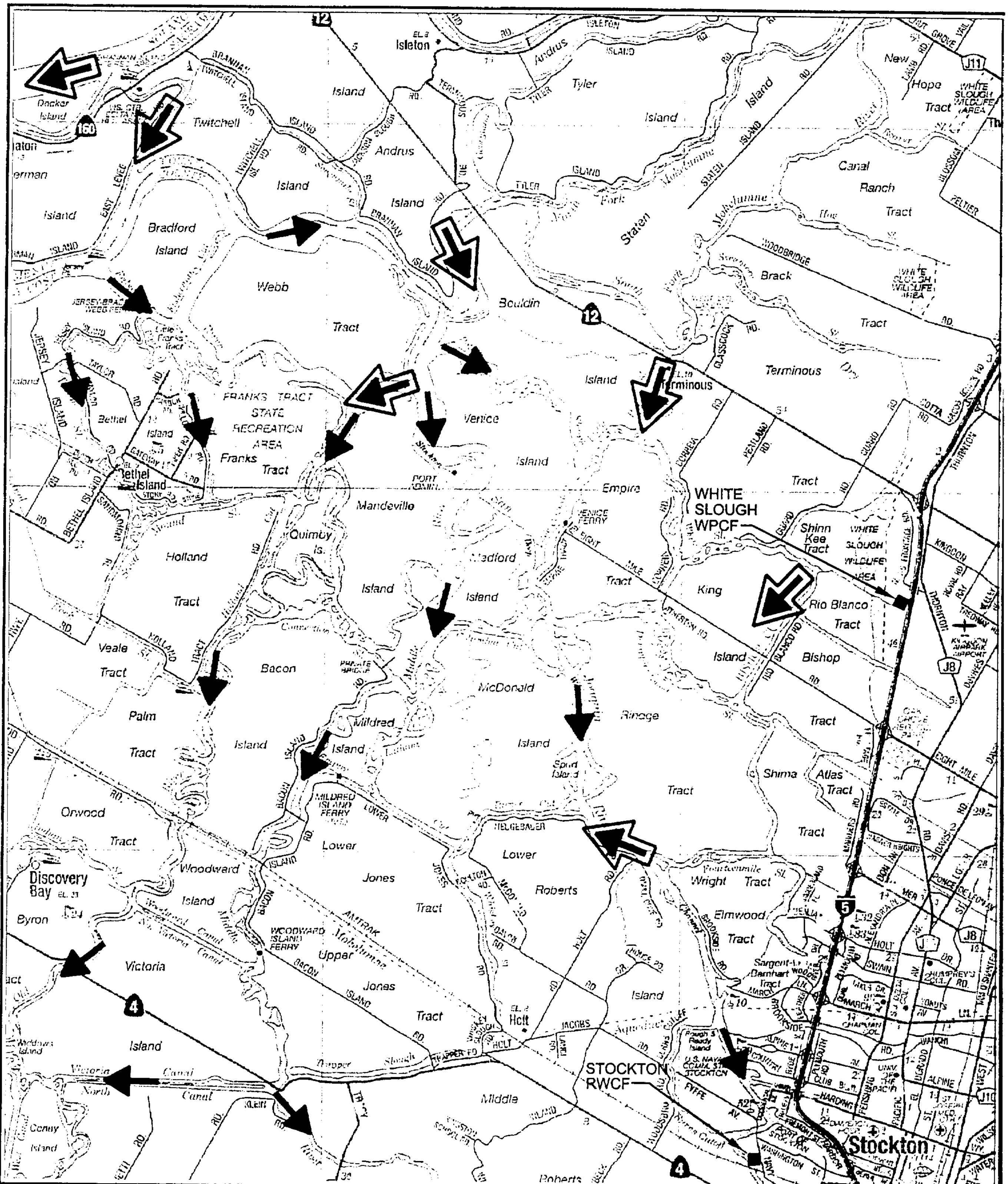


NOTES:

1. SOURCE OF UNDERLYING MAPPING SHOWN HEREON IS A SCANNED MAP
2. BISHOP CUT FLOW DIRECTION FROM MODELING BY DR. GARY LITTON

LEGEND:

POSITIVE FLOW

REVERSE FLOW

N

NOT TO SCALE

Figure 4-2

City of Lodi

Wastewater Master Plan

DELTA CIRCULATION PATTERNS

LOW FLOWS / HIGH EXPORT

WEST YOST & ASSOCIATES

Table 4-1. Current Interim Requirements for Discharge of Treated Municipal Effluent – Major Parameters

Constituent or Parameter	Units	Limit
BOD	mg/L	20/40/50[(a)]
TSS	mg/L	20/40/50[(a)]
Total Coliform	MPN/100 mL	23
Acute Toxicity	Survival one/three	70%/90%
Chronic Toxicity	TUc	1
Dissolved Oxygen	mg/L	5.0 minimum

[(a)] Monthly average/weekly average/daily maximum.

Effluent from the Water Pollution Control Facility (WPCF) consistently complied with the previous discharge requirements for BOD, TSS, and toxicity. There were three instances in 1996 and one instance in 1999 when individual biotoxicity test results were outside the allowable survival rate, but the adverse results did not occur in consecutive tests so as to cause a violation of the permit requirements. The suspected cause for the instances of toxicity in 1996 was excessive use of sulfur dioxide for dechlorination.

Reclamation Requirements

The City irrigates animal feed crops on its own land surrounding the treatment plant using a mixture of non-disinfected secondary effluent, digested biosolids, and industrial (mostly cannery) wastewater. The current discharge requirements for the secondary effluent are 40 mg/L BOD and 0.2 mL/L settable matter (SM) (monthly averages). The current discharge requirements also contain other operational restrictions derived from Title 22, Division 4 Reclamation Requirements or Department of Health Services guidelines.

The reclamation requirements state that nutrient loading of the reclamation area shall not exceed the crop demand. The City's nitrogen loading rates have been consistently below agronomic use rates. However, nitrate concentrations in several of the shallow groundwater monitoring wells have exceeded the 10 mg/L drinking water standard. The causes of the relatively high nitrate levels have not been determined.

Solids Disposal/Reuse Discharge Requirements

Biosolids disposal and reuse practices are required to conform with Section 405(d) of the Federal Clean Water Act. In addition, nitrogen loading rates from biosolids are included in the total reported nitrogen loadings for the City's land. Total nitrogen loading rates are not allowed to exceed crop uptake and denitrification rates in order to protect groundwater quality.

Industrial Wastewater Discharge Requirements

Because the industrial wastewater is applied directly to the land, there are no specific effluent quality requirements. The main requirements are related to the prevention of odors and groundwater impacts.

Receiving Waters Modeling

A dilution study of White Slough and Bishop Cut receiving waters was performed by Whitley Burchett & Associates in 1994. The average dilution ratio over the tidal cycle at the confluence of White Slough and Bishop Cut (monitoring point R-2, see Figure 4-1) was estimated to be approximately 8:1 for an effluent flow of approximately 6 Mgd.

A more detailed model of Dredger Cut, White Slough, and Bishop Cut was completed in 1998 by Dr. Gary Litton and Jason Nikaido at the University of the Pacific.[2] The average dilution in Dredger Cut was estimated to be 2:1 for an 8.5 Mgd effluent flow rate. The average dilution at the east side of the confluence of Dredger Cut and White Slough (R-2) was estimated to be 4:1.

Sampling and modeling dissolved oxygen concentrations within Dredger Cut were the main focus of the Litton study. One of the most significant results was that dissolved oxygen (D.O.) levels in Dredger Cut dropped below 5 mg/L on several occasions during the testing period even when the treatment plant was not discharging, indicating impacts from other non-point sources of pollution. The dissolved oxygen model predicted that treatment plant effluent with 20 mg/L BOD would cause D.O. levels in Dredger Cut to drop below 5 mg/L at low slack tides. At an effluent BOD concentration of 10 mg/L, the D.O. concentration was predicted to remain above 5 mg/L at low slack tides assuming inputs from non-point pollutions sources were not severe.

Subsequent to the Dredger Cut study, Dr. Litton performed modeling of net flows in White Slough and Bishop Cut using models from the Department of Water Resources and CALFED[3]. The model results showed that for an 8.5 Mgd discharge, the average annual dilution in Bishop Cut would be 12:1. The highest dilutions would be in December through March, with at least 20:1 dilution in half the years and at least 10:1 dilution in about 96 percent of the years. The lowest dilutions would occur in May through July, with an average dilution of only 6:1. Some of the potential future modifications to Delta channels proposed by CALFED would reduce dilutions by 20 to 70 percent. See Appendix [redacted] for more details.

POTENTIAL CHANGES TO DISCHARGE LOCATION AND BENEFICIAL USES

Discharge to Bishop Cut

Construction of an outfall pipeline or channel to Bishop Cut immediately south of the confluence with White Slough is a potential alternative for providing improved effluent dilution flows. Water quality objectives for the receiving water would be easier to meet with more dilution. A diffuser across the most active portion of the channel would provide an estimated dilution of over 20:1 during most winter months, but only about 6:1 during summer months.

Sports Complex

A sports complex has been proposed for 400 acres in the southeastern portion of the City's property. This complex would include a significant portion of grass fields which would need irrigation. The current project concept calls for peak summertime use of up to 2.5 Mgd of treated effluent meeting Title 22, Division 4 Reclamation Requirements for unrestricted irrigation as the irrigation water source for the fields.

FUTURE DISCHARGE REQUIREMENTS

General

New waste discharge requirements will be enforced when the interim requirements expire in April 2004. For discussion purposes, these anticipated new waste discharge requirements are referred to in this report as "Year 2004 discharge requirements". The City would have to apply for new discharge requirements if the point of discharge were changed. Probable requirements resulting from an application for a new point of discharge are referred to as "anticipated discharge requirements". Requirements which may be imposed in future permits are referred to as "potential future discharge requirements". Anticipated and potential future discharge requirements presented in this report were developed from discussions with Regional Board staff, current discharge requirements, and the review of relevant research and guidelines.

Municipal Effluent Discharge to Dredger Cut

Discharge to Dredger Cut will need to satisfy current, Year 2004, and future discharge requirements mandated by the EPA and Regional Water Quality Control Board. The most significant new requirements are related to trace toxins, dissolved oxygen objectives, disinfection, and biosolids reuse. Current interim, Year 2004, and potential future discharge requirements are listed in Table 4-2 along with average and peak values from the last 5 to 10 years for comparison purposes. The anticipated and future discharge requirements include no dilution in Dredger Cut for water quality objectives. The enlarged bold values are those likely to be difficult to meet with current facilities. Complete results from the City's trace toxins sampling program since December 1992 are shown in Appendix [redacted].

Discharge requirements shown in Table 4-2 are based on meeting Delta water quality objectives at Location R-1 in Dredger Cut. BOD requirements are effectively dictated by the D.O. objective for Dredger Cut. As discussed previously, modeling indicates that the 5 mg/L D.O. requirement cannot be reliably met for effluent with BOD above 10 mg/L.

Contact recreation and agricultural irrigation are listed in the Basin Plan as beneficial uses of the Delta. The Year 2004 and potential future disinfection requirements incorporate the general recommendation from the Department of Health Services (DHS) that discharges to streams with little dilution should be tertiary treated to the same levels as required for unrestricted irrigation water as per wastewater reclamation requirements contained in Title 22, Division 4 of the California Water Code. This includes coagulation, sedimentation, and filtration or a DHS-approved direct filtration alternative process. Previous DHS guidelines have not recommended tertiary treatment when dilution in the receiving waters was above 20:1.

Table 4-2. Current Interim, Year 2004, and Potential Future Discharge Requirements For Discharge to Dredger Cut[a]

Constituent or Parameter	Units	Current Interim	Year 2004	Potential Future	Historical Average	Historical Peak
BOD	mg/L (30 day)	20	**10**	**10**	8.4	16
TSS	mg/L (30 day)	20	**10**	**10**	10.0	24
D.O.	mg/L (receiving water)	**5**	**5**[b]	**5**[b]	5.2	0.6 (min)
Temperature	Δ°F (receiving water)	**5**	**4**[b]	**4**[b]	9.3	21.6
Chlorine Residual	mg/L	0.01	0.01	0.01	<0.1	4.6
Coliform	MPN/100 mL	23	**2.2 filtered**[c]	**2.2 filtered**[c]	2	13[d]
Lead	µg/L	n/a	**3.7**	**3.7**	<5 (total)	10 (total)
Zinc	µg/L	n/a	**101**	**100**[b]	105 (total)	160 (total)
Cyanide	µg/L	n/a	5.2	5.2	<10	49
Mercury	µg/L	n/a	TML	**0.050**[e] **& TML**	<0.2	0.63
Bis-2 ethyhexyl phthalate	µg/L	n/a	n/a	**11.8**[e]	<15 (median)	190
Chloroform	µg/L	n/a	n/a	**10.4**[e]	21	102
Lindane	µg/L	n/a	**n.d. (0.02)**	**n.d.**	0.01 (median)	0.051
Chronic Toxicity	TUc	10	**1**	**1**	1 (median)	>16
Acute Toxicity	% survival	70/90	70/90	70/90	99.2	85 (min)
Ammonia	mg/L	n/a	n/a	**3.2**[f]	1.2	6.5
Total Nitrogen	mg/L	n/a	n/a	**TML**[g]	9.4	—
Total Phosphorous	mg/L	n/a	n/a	TML[g]	0.21	—

[a] All limits are for average concentrations, typically a 30-day averaging period.
[b] Basin Plan. (Limits for metals expressed as dissolved concentrations.)
[c] Proposed DHS/Regional Board guidelines, may be incorporated into future Basin Plan.
[d] Monthly median, 9 days have exceeded 500 MPN/100mL since Jan 1994.
[e] EPA California Toxics Rule, metals limits expressed as dissolved concentrations.
[f] EPA Ambient Water Quality Criteria, no dilution assumed for chronic criteria.
[g] No specific requirements pending, Total Mass Loads (TML) may be applied in the future.

The potential for nutrient mass limits in the future is based on the fact that Total Mass Daily Loadings are being proposed for Stockton and other dischargers who may contribute to the dissolved oxygen sag in the Stockton Deepwater Ship Channel. The current proposals only address BOD limits, but excess nutrients are recognized as contributors to the problem. Lodi's discharge only appears to impact the lowermost reach of the Deepwater Ship Channel under high export conditions. This reach below (northwest of) Turner Cut does not experience dissolved oxygen sags which violate Delta water quality objectives[4] (see Figure 4-2). However, it would be prudent to begin considering the possibility of nutrient limitations in long term planning.

Compliance with Year 2004 Requirements. The treatment plant was designed to produce an effluent with a BOD concentration of 20 mg/L at 8.5 Mgd without nitrification. The WPCF has historically produced effluent with an average BOD of less than 10 mg/L and essentially all ammonia converted to nitrate (full nitrification). There have been a few recent instances when the City had difficulty achieving full nitrification, so it appears that the plant reached its original nitrification capacity limit at approximately 6.0 Mgd. Disinfection and biotoxicity test results could be adversely affected if the treatment plant cannot fully nitrify. Reliably achieving 10 mg/L BOD could also become more difficult as the plant approaches its 8.5 Mgd original design capacity.

Since the treatment process does not currently include filters and a chlorine contact tank sized for tertiary treatment, meeting Title 22, Division 4 treatment and disinfection requirements would not be possible without additional facilities. Some anticipated discharge requirements related to trace toxins may be difficult to consistently meet. The plant effluent has contained concentrations of zinc ranging up to 160 mg/L (as total recoverable metal). This would be in excess of the Year 2004 discharge limits for zinc, although the effective limit could be reduced, depending upon the relationship between total and dissolved zinc for the treatment plant effluent. The plant effluent contained cyanide in excess of the anticipated limit on two occasions in 1995 and one occasion in 1996. The new lead requirement could be exceeded on occasion.

During winter months, the plant effluent is considerably warmer than the water in Dredger Cut. Permit requirements specify that the surface water temperature cannot be raised by more than 4°F at any location. While it is unlikely that aquatic life is adversely affected by the warmer water temperature near the discharge, there could be a technical violation of the temperature requirement.

Compliance with Potential Future Requirements. The potential future requirements in Table 4-2 which are more restrictive than the anticipated discharge requirements are the requirements for ammonia, mercury, chloroform, and nutrients. Although there has been only one sampling result which contained detectable mercury, the detection limit for mercury (0.20 μg/L) was higher than EPA ambient water quality criteria for chronic toxicity (0.012 μg/L). Based on effluent quality measurements to date, meeting potential future requirements for mercury, zinc, and nitrogen would not be possible with existing treatment facilities. Based on limited data, the secondary treatment process as it is currently operated produces effluent with unusually low levels of phosphorus, which would probably satisfy future TMLs.

Chloroform and other trihalomethanes are formed as byproducts of chlorine disinfection. There are no established diversions for drinking water use in the northwestern portion of the Delta. It is unclear what mixing zone and dilution would be allowed for this water quality objective since it is intended to protect sources of drinking water rather than aquatic life. Assuming no dilution in Dredger Cut, this potential requirement would be very difficult to meet with existing facilities. If dilution beyond Dredger Cut were allowed to be considered, the chloroform objective could probably be satisfied.

Municipal Effluent Discharge to Bishop Cut

As discussed previously, one of the obvious alternatives for the City is to construct an outfall to Bishop Cut. This would provide more dilution for meeting receiving water quality objectives. In addition, water at R-3 in Bishop Cut (see Figure 4-1) has always contained dissolved oxygen substantially above the 5.0 mg/L water quality objective for the Delta based on monitoring by the City. Taking the greater available dilution into account, the near-term anticipated and potential future discharge requirements are listed in Table 4-3. Anticipated and potential future effluent limits shown for trace toxins are based on either an assumed 5:1 average dilution and continuous concentration criteria or maximum concentration criteria, whichever is more restrictive. Values shown in enlarged bold are those likely to be difficult to meet with current facilities. Diurnal storage of effluent could be required so that discharge does not occur at low dilution ratios during slack portions of the tidal cycle.

Compliance with Anticipated Requirements. For treated effluent discharged directly to Bishop Cut during times when dilution exceeds 20:1, effluent quality similar to that achieved historically should be adequate to satisfy anticipated discharge requirements, except possibly for zinc concentrations. However, during most of the year, the more stringent BOD, TSS, and disinfection requirements could not be met without new tertiary treatment facilities. Tertiary treatment may or may not be required for Winter months at less than 20:1 dilution, depending upon the interpretation of law, policies, and site-specific conditions. There may be some difficulty achieving consistent disinfection results as flows increase, especially if nitrification cannot be assured throughout the year.

Compliance with Potential Future Requirements. Disinfection requirements could become more stringent in the future depending upon actual dilution ratios in Bishop Cut. Total mass limits could be adopted for BOD, mercury, and nutrients in the future.

Municipal Effluent Reuse—Unrestricted Irrigation

The anticipated discharge requirements for unrestricted irrigation of fields at the proposed Sports Complex or food crops are shown in Table 4-4. These requirements generally reflect standard Reclamation Requirements from Title 22, Division 4 of the Water Code. New tertiary filtration treatment facilities would be required to satisfy these requirements.

Municipal Effluent Reuse—Animal Feed Crops

Discharge requirements for irrigation of animal feed crops are not anticipated to change substantially in the future. These are shown in Table 4-5.

The anticipated and future potential requirements for animal feed crop irrigation should be easy to satisfy with existing treatment processes. Effluent disinfection could potentially be required to satisfy future site specific concerns regarding potential public or farm worker contact with the effluent.

Table 4-3. Anticipated and Potential Future Discharge Requirements For Discharge to Bishop Cut

Constituent or Parameter	Units	Near Term Anticipated	Potential Future	Historical Average	Historical Peak
BOD	mg/L (30-day)	**10**[(a)]	**TML**	8.4	16
TSS	mg/L (30-day)	**10**[(a)]	**10**[(a)]	10.0	24
D.O.	mg/L (receiving)	5[(b)]	5[(b)]	9.3	2.9 (min)
Temperature	Δ°F (receiving)	4[(b)]	4[(b)]	9.3	21.6
Chlorine Residual	mg/L	0.01[(c)]	0.01[(c)]	<0.1	4.6
Coliform	MPN/100 mL	**2.2**[(a,d)] **filtered**	**2.2**[(d)]**filtered**	2	13
Zinc	µg/L (receiving)	100[(b)]	100[(b)]	105 (total)	160 (total)
Zinc	µg/L (effluent)	**101**[(b)]	**101**[(b)]	105 (total)	160 (total)
Cyanide	µg/L (receiving)	10[(b)]	10[(b)]	<10	49
Cyanide	µg/L (effluent)	22[(e)]	22[(e)]	<10	49
Mercury	µg/L (receiving)	**0.050**[(e)]	**TML**	<0.2	0.63
Mercury	µg/L (effluent)	1.4[(e)]	1.4[(e)]	<0.2	0.63
Bis-2 ethyhexyl phthalate	µg/L	n/a	118[(e)]	<15 (median)	190
Chloroform	µg/L	n/a	**29**[(e)]	21	102
Lindane	µg/L (receiving)	0.02	n.d.	0.01 (effluent)	0.051 (effluent)
Chronic Toxicity	TUc	5[(b)]	5[(b)]	1 (median)	>16
Acute Toxicity	% survival	70/90	70/90	99.2	85 (min.)
Ammonia	mg/L	n/a	6	1.2	6.5
Total Nitrogen	mg/L	n/a	**TML**[(f)]	9.4	—
Total Phosphorous	mg/L	n/a	TML[(f)]	0.21	—

(a) BOD and TSS limits of 30 mg/L, coliform limit of 23 MPN/100mL may be allowed during Winter months and/or when dilution exceeds 20:1.
(b) Basin Plan. (Limits for metals expressed as dissolved concentrations.)
(c) EPA Ambient Water Quality Criteria (imposed through Basin Plan narrative toxicity requirements).
(d) Proposed DHS/Regional Board guidelines, may be incorporated into future Basin Plan.
(e) California Toxics Rule.
(f) No specific requirements pending, future Total Mass Limits may apply.

Table 4-4. Anticipated Discharge Requirements for Unrestricted Irrigation

Constituent or Parameter	Units	Anticipated
BOD	mg/L	**10**
TSS	mg/L	**10**
Turbidity	NTU	**2**
Coliform	MPN/100 mL	**2.2 filtered**
Ammonia + Nitrate	lbs/ac/yr	Agronomic use

Table 4-5. Current and Potential Future Discharge Requirements For Irrigation of Animal Feed Crops

Constituent or Parameter	Units	Current	Potential Future
BOD	mg/L	40	30
SM	ml/L	0.2	0.1
Coliform	MPN/100 mL	Secondary	23
Ammonia + Nitrate	lbs/ac/yr	Agronomic use	Agronomic use

Industrial Effluent Irrigation Reuse

The industrial wastewater is principally from the Pacific Coast Producers (PCP) cannery. The main discharge requirements for industrial wastewater involve the prevention of nuisance odors and adverse impacts to groundwater. Current, anticipated, and potential future requirements are listed in Table 4-6.

Table 4-6. Current and Potential Future Discharge Requirements For Irrigation with Industrial Wastewater

Constituent or Parameter	Units	Anticipated	Potential Future
BOD	lbs/ac/day	n/a	200
Hydrogen Sulfide	mg/L	n/a	**1.0**
Dissolved Oxygen	mg/L	n/a	**1.0 minimum**
Salinity	lbs/ac/yr	No significant impacts	No significant impacts
Ammonia + Nitrate	lbs/ac/yr	Agronomic use	Agronomic use

Distribution facilities may need some improvements to minimize the potential for sulfide generation and odors from industrial wastewater irrigation. Average fixed mineral TDS for the industrial effluent is approximately 800 mg/L *vs.* 400 to 500 mg/L for the municipal effluent. The industrial wastewater would be considered good quality for irrigation and should not cause significant impacts to groundwater. A zero degradation objective applied to major mineral constituents is a future possibility. It would be nearly impossible to meet if strictly interpreted and applied to shallow groundwater directly under the irrigation fields.

Biosolids Disposal/Reuse

The City currently produces approximately 320 metric tons (dry weight basis) of biosolids annually. The existing anaerobic digesters and lagoon produce Class "B" biosolids under the new Federal 40 CFR Part 503 regulations. The biosolids are mixed with the irrigation water and applied via surface irrigation to land designated for annual row crops (approximately 300 acres irrigation in any one year). A total of 600 acres (243 ha) is used for biosolids application on a multi-year rotation. The discharge requirements for biosolids are derived from the Federal Part 503 regulations and the proposed General Biosolids Permit authored by the State Water Resources Control Board. These requirements generally address maximum concentrations and loading rates for heavy metals and operational procedures to prevent pathogen transmission. The maximum concentrations and loading rates for metals and other constituents under the Part 503 regulations are included in Table 4-7. The proposed General Biosolids Permit is not applicable to areas in the statutory Delta, but many of the operational requirements from the General Biosolids Permit will undoubtedly be applied to Lodi's site specific permit.

Table 4-7. Biosolids Limits

Constituent	Ceiling Concentration, mg/kg	Max. Cumulative Loading, kg/ha	Historical Concentration, mg/kg	Average Loading, kg/ha/yr	Life of Existing Site, years
Arsenic	75	41	7.8	0.01	4,100
Cadmium	85	39	5.6	0.007	5,600
Chromium	3,000	3,000	22	0.029	10,400
Copper	2,500	1,500	246.0	0.32	4,700
Lead	350	300	30.5	0.04	7,500
Mercury	57	17	5.5	0.007	2,400
Molybdenum	75	18	11.1	0.014	—
Nickel	420	420	15.0	0.019	22,000
Selenium	100	100	1.2	0.002	50,000
Zinc	7,500	2,800	604.0	0.80	3,500
Total N (lbs/ac/yr)	Agronomic use	Agronomic use	—	—	—

Compliance with Biosolids Limits. The biosolids limits should be reasonably easy to comply with as long as sufficient land continues to be available for biosolids application. The distribution uniformity of biosolids may have to be improved to effectively utilize all available land.

SUMMARY AND CONCLUSIONS

Discharge to Dredger Cut will require more highly treated effluent than is reliably obtainable with current facilities. Compliance with dissolved oxygen, disinfection, and zinc requirements will be problematic. Potential future requirements for other trace toxins and nutrients may also be impossible to meet with current facilities.

Requirements for discharge to Bishop Cut could possibly be satisfied during Winter months using existing treatment processes with the addition of capacity for full nitrification and a reduction in peak zinc concentrations. Tertiary treatment will probably be required for discharge to Bishop Cut during Spring through mid-Fall months. Delta channel modifications by CALFED could necessitate year-round tertiary treatment.

Land application and irrigation reuse of effluent on animal feed crops would have the least restrictive treatment requirements. Landscape irrigation or irrigation of food crops would require compliance with Title 22 Reclamation requirements, including tertiary filtration and disinfection.

Dilution flows and dissolved oxygen impacts in Bishop Cut should continue to be evaluated in greater detail for a discharge into the west portion of Bishop Cut immediately south of the confluence with White Slough. The potential impacts of BOD in downstream Delta channels should also be evaluated.

REFERENCES

[1] State Water Resources Control Board. *Water Quality Control Plan for the San Francisco Bay/Sacramento-San Joaquin Delta Estuary*. Pub. # 95-1WR, May 1995.

[2] Litton, G.M. and J. Nikaido. *Water Quality Impact Report White Slough Water Pollution Control Facility*. Draft, University of the Pacific, Department of Civil Engineering, October 1998.

[3] Litton, G.M.. *Draft City of Lodi Outfall Relocation Study*. May 9, 2000.

[4] Jones and Stokes Associates. *Potential Solutions for Achieving the San Joaquin River Dissolved Oxygen Objectives*. Prepared for DeCuir and Somach and the City of Stockton. June 1998.

SECTION 5. ALTERNATIVES FOR SATISFYING DISCHARGE REQUIREMENTS

INTRODUCTION

The purpose of this section is to identify general alternatives for satisfying the anticipated and potential future discharge requirements. These are broad process or disposal/reuse alternatives targeted to discharge requirements listed in Section 4 as difficult to meet with existing facilities. The alternatives identified in this section will be evaluated in detail in later sections.

ALTERNATIVES FOR SATISFYING MUNICIPAL EFFLUENT DISCHARGE REQUIREMENTS

There are a number of discharge, treatment process, and land treatment or reuse alternatives which could partially or fully enable the City to meet the anticipated and potential future discharge requirements. The most plausible alternatives and their anticipated effectiveness in meeting requirements are discussed in this section.

The discharge requirements listed in Table 5-1 are those which will be difficult or impossible to meet with existing facilities. The relative effectiveness of the alternatives at meeting the requirements is indicated by 0 through 3 stars as described in the table legend. The effectiveness of combinations of alternatives is generally additive.

Surface Discharge Alternatives

Continued Discharge to Dredger Cut. The City could continue to discharge to Dredger Cut as it currently does. The lack of net dilution in Dredger Cut effectively subjects the City to stringent discharge requirements for most constituents and parameters. Factors outside the City's control, such as low dissolved oxygen levels, would prohibit discharge at some times.

Bishop Cut Outfall Pipeline. Flows in Bishop Cut are strongly influenced by tidal cycles. Water flows east up White Slough and south down Bishop Cut during a rising tide. Flows reverse during an ebbing tide. Peak instantaneous flows over a tidal cycle are about 600 cfs. In addition, there is a net flow south in Bishop Cut which coincides with the general north-south net flows across the Delta to the major diversions in the South Delta. The net flow averages about 160 cfs. Net flows are lower in summer months and are low when the State Water Project and Central Valley Project pumps are shut down for a couple weeks in Spring due to the fish migrations. Estimated net flows during these periods are about 50 to 100 cfs. Future modifications to Delta channels could also reduce net flows to Bishop Cut. Additional details are contained in the City of Lodi Outfall Relocation Study completed in May 2000 (Appendix [redacted]).

Construction of a 1.3-mile pipeline, as shown in Figure 5-1, to move the discharge to the north end of Bishop Cut would provide an estimated dilution of between 5:1 and 40:1 for the treated effluent. In combination with full nitrification treatment, this should allow the City to meet

Table 5-1. Alternatives for Satisfying Municipal Effluent Discharge Requirements[a]

Alternative	Discharge Requirements									Type of Alternative		
	Year 2004 and Anticipated						Possible Future			WPCF Process	Discharge	Land Trt. Or Reuse
	DO[b]	BOD	Temp.	Bact.	Zinc	NH3	Hg	N	P			
Bishop Cut Outfall Pipeline	★★★	★★	★★	★	★		★				✓	
Bishop Cut Outfall Wetlands	★★★	★★	★★★	★	★★		★★	★★			✓	✓
Nitrification		★				★★★				✓		
Tertiary Filtration and Disinfection		★		★★★	★		★			✓		
Biological N Removal								★★		✓		
Biological P Removal									★★	✓		
Chemical P & Heavy Metals Removal					★★				★★★	✓		
Other Treatment Wetlands			★★★		★★		★★	★★				✓
Winter Percolation	★★★	★★★	★★★	★★★	★★★	★★	★★★	★	★★★			✓
Winter Well Injection	★★★		★★★		★★		★					✓
Additional Storage	★		★★									✓
Source Control					★		★					

(a) Legend:

- ★ Helps meet anticipated and future requirements.
- ★★ Meets anticipated requirements and probably meets future requirements.
- ★★★ Reliably meets anticipated and future requirements.

(b) DO (dissolved oxygen) refers to the DO of receiving waters. Discharge requirements prohibit discharge when receiving water DO is below 5 mg/L. This occurs in Dredge Cut even when the WPCF is not discharging.

anticipated discharge requirements for all parameters except zinc when dilution is over 20:1. When dilution is under 20:1, tertiary filtration and disinfection are likely to be required. The extra dilution would help the City meet potential future mercury concentration limits, but potential future mass loading limits for nutrients, mercury, or BOD could be problematic.

The discharge requirements specify a maximum temperature differential of 20 degrees Fahrenheit, a maximum surface water temperature change of 4 degrees Fahrenheit, and a maximum discharge zone temperature change of 1 degree Fahrenheit over 25 percent of the channel. The maximum temperature differential between treated effluent and Bishop Cut has been 22 degrees Fahrenheit. If effluent is briefly stored in the storage ponds and discharged through a well-designed diffuser when flows in Bishop Cut are over 100 cfs, the temperature requirements should be achievable. The city's existing storage ponds could be used to provide short-term storage during the slack phases of tidal cycles.

Bishop Cut Outfall Wetlands. This alternative combines the benefits of extra dilution from Bishop Cut discharge with polishing treatment from a constructed wetlands. For this alternative, wetlands would be constructed between the existing discharge into Dredger Cut and a new discharge point at the north end of Bishop Cut, thereby eliminating the need for an outfall pipeline. The wetlands would provide better temperature equalization and removal of some zinc, mercury, nitrate, and any chlorine residual. The wetlands would also provide a significant amount of storage and effluent disposal/reuse capacity. Wetlands provide some pathogen removal, but tertiary treatment would probably still be required when dilution is less than 20:1.

WPCF Treatment Process Alternatives

Nitrification (Ammonia Removal). The treatment plant was not designed to provide ammonia removal, however it has proven to be capable of reliably removing ammonia up to a flow rate of about 6 Mgd. With recently implemented changes to the air diffusion system, the capacity of the treatment plant at full nitrification is approximately 7 Mgd. Providing treatment capacity for the conversion of essentially all ammonia to nitrate at the 8.5 Mgd Master Plan flow rate would help satisfy biotoxicity discharge requirements. Nitrification also helps to reduce dissolved oxygen uptake in the receiving waters and improve the reliability of chlorine disinfection. If direct ammonia limits are imposed in the future, nitrification would be mandatory.

Tertiary Filtration and Disinfection. Coagulation, sedimentation, and filtration (or DHS-approved equivalent) followed by enhanced disinfection is required for discharge to Dredger Cut in the new Waste Discharge Requirements. Tertiary filtration and disinfection would probably also be required for discharge to Bishop Cut during at least the warm months of the year. Along with satisfying tertiary treatment requirements, filters would make it much easier to reliably meet a 10 mg/L BOD effluent discharge requirement. Filters would also help reduce TSS and heavy metals, especially if used in conjunction with the addition of aluminum or iron salts to aid coagulation.

Biological Nitrogen Removal. Nitrate can be converted to nitrogen gas when there is adequate carbon in an anoxic zone in the treatment process or in percolation basin soils. This would help satisfy potential future loading limits on nitrogen discharged to surface waters. It would also help satisfy limitations on nitrate loading to groundwater under fields or percolation basins.

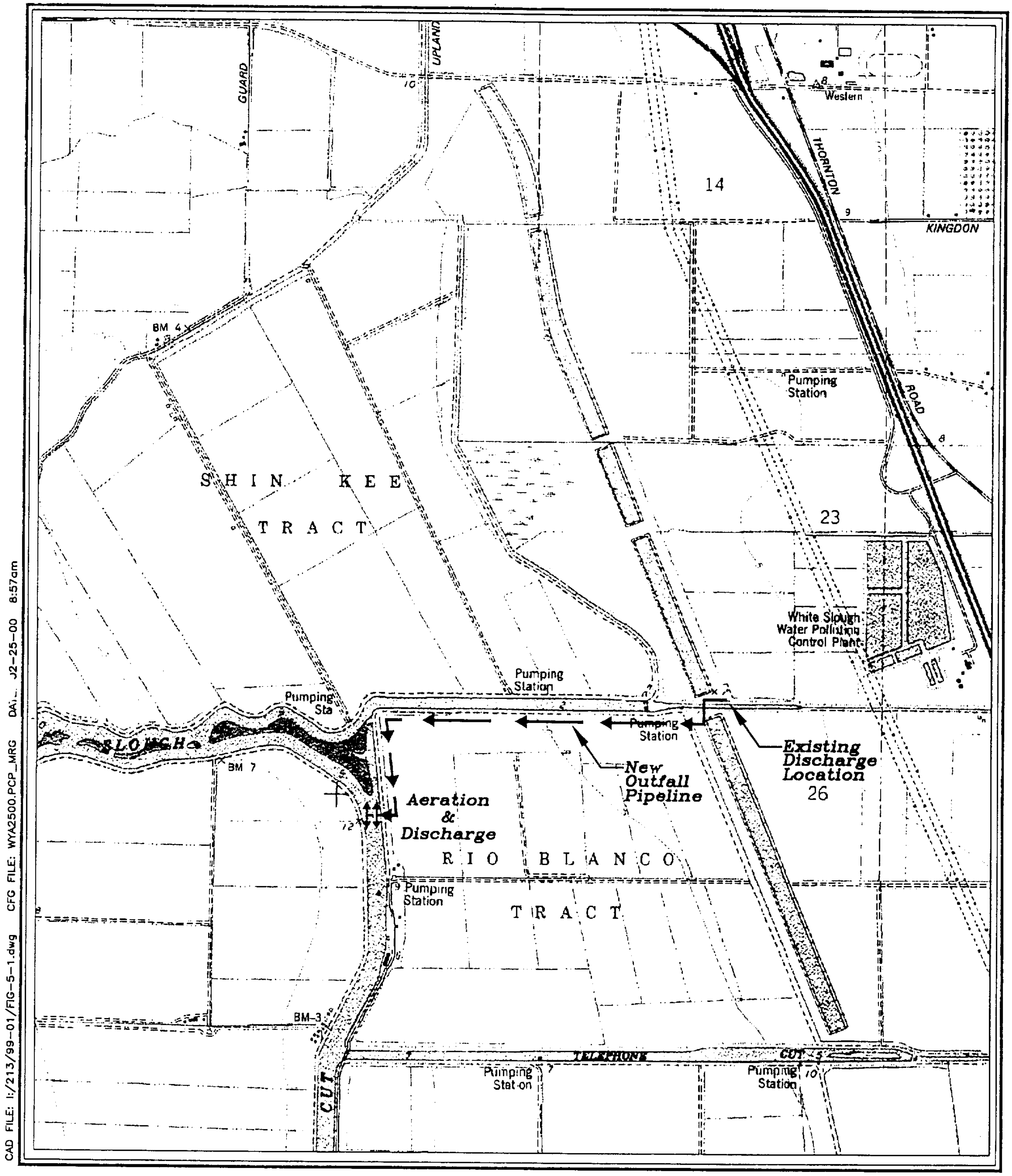


Figure 5-1

City of Lodi
Wastewater Master Plan

Bishop Cut Outfall
Pipeline Alternative

WEST YOST & ASSOCIATES

Biological Phosphorous Removal. Biological uptake of phosphorous is an unintended feature of the current secondary treatment process of the WWTP through the use of an anaerobic selector zone. The successful operation of an anaerobic selector may need to be continued to help satisfy potential future loading limits on phosphorous.

Chemical Phosphorous and Heavy Metals Removal. The addition of aluminum or iron salts into or after the secondary process would remove phosphorous and particulate associated heavy metals through precipitation and enhanced removal of suspended solids.

Land Treatment and Reuse Alternatives

Wetlands Treatment. Wetlands could be constructed on land near the treatment plant. Wetlands could provide polishing treatment for secondary effluent from the treatment plant, including temperature equalization and removal of zinc, mercury, nitrate, and chlorine residual. The wetlands would also provide additional storage and effluent disposal/reuse capacity.

Winter Percolation Disposal. Some of the soils in the area are relatively sandy with moderately high permeability. It may be possible to utilize fields with little or no slope for percolation disposal of effluent during late autumn and winter months. Improvements in field leveling, groundwater level control, runoff-return systems, and/or containment berms would be required for this alternative. If successful, this alternative would eliminate discharge to surface waters and all the associated discharge requirements. The major remaining concern would be a potential increase in nitrate loadings to groundwater. This could be minimized through percolation disposal operational techniques and/or treatment process modifications. Additional land would need to be purchased or leased with this alternative.

Winter Well Injection/Saltwater Intrusion Barrier. Another alternative for winter disposal could be well injection of tertiary treated effluent to prevent saltwater infusion from the west. This is currently being practiced in Orange County. Significant geological and funding issues would need to be addressed before this alternative could be compared with other alternatives.

Additional Storage. If the surface water discharge point continues to be in Dredger Cut, all effluent will have to be stored or reused during periods of time when the dissolved oxygen level in Dredger Cut falls near or below 5.0 mg/L. Storage ponds could also reduce the temperature of effluent during winter months so that temperature related discharge requirements could be satisfied. Some additional effluent storage may be desirable to provide flexibility for properly managing irrigation reuse operations.

Source Control

Source control will need to be an element of the City's wastewater management program regardless of the final disposal location chosen for the effluent. Current water quality control legislation requires that dischargers take all reasonable measures to reduce the concentrations of trace toxins regardless of whether or not an actual problem has been observed. Since the concentrations of these constituents are so low, end of pipe treatment systems are costly and often of little effectiveness. For purposes of this report, however, efforts substantially beyond traditional source control measures are listed as "source control" for the reduction of zinc concentrations. It may be possible to identify some of the sources of zinc and other heavy metals through upstream sampling programs. There is some preliminary data indicating that elevated levels of zinc may be coming from older areas of town (galvanized water pipes) and from industrial sources. If the sources can be positively identified, the concentrations of zinc and other metals could be reduced through changes at industrial sources or water system operational changes. Water system operational changes could include minimizing the run time of the wells which produce the most aggressive water as measured by the Langlier Index. Other potential changes could include adding calcium or phosphate compounds at wellheads to reduce corrosion of galvanized pipes.

Mercury levels may be reduced as a result of the source identification and reduction program recently initiated by the City. Current levels may be reduced through the installation of amalgam recovery systems in district offices and florescent tube and thermometer collection program.

Combinations of Alternatives to Satisfy Requirements

Combinations of discharge, process, and/or land treatment alternatives will be needed to satisfy all the anticipated and potential future discharge requirements listed in Table 5-1. The apparent reasonable combinations of alternatives are listed in Table 5-2.

Table 5-2. Apparent Reasonable Combinations of Alternatives

Combination	Discharge Point	Facilities Needed for Anticipated Requirements	Possible Facilities Needed for Potential Future Requirements[a]
DC-D	Dredger Cut – Direct	Additional Storage Nitrification Tertiary Treatment Source Control	Biological N Removal Biological P Removal Metal Salts Addition
DC-W	Dredger Cut – Wetlands	Additional Storage Nitrification Tertiary Treatment Wetlands	Biological P Removal
BC-D	Bishop Cut – Direct	Outfall Pipeline Nitrification Tertiary Treatment Source Control	Biological N Removal Biological P Removal Metal Salts Addition
BC-W	Bishop Cut – Wetlands	Outfall Wetlands Nitrification Tertiary Treatment	Biological P Removal
BC-PD	Bishop Cut – Partial Discharge	Percolation Basins and Fields Nitrification Biological N Removal[b] Source Control	Biological P Removal Metal Salts Addition
BC-PW	Bishop Cut – Partial Discharge with Wetlands	Percolation Basins and Fields Nitrification Biological N Removal[b]	Biological P Removal

LD	Land Discharge	Percolation Basins and Fields Nitrification[c] Biological N Removal[b]	

[a] The existing plant already has excellent biological P removal even though it was not specifically designed for P removal. This process feature may be useful for satisfying future requirements and should be specifically considered in capacity expansion alternatives.

[b] Wetlands could be substituted for biological denitrification in treatment process tanks. Some denitrification can occur in the percolation basin soils.

[c] Nitrification may not be mandatory depending upon percolation basin design and operation.

Alternatives for seasonal discharge of secondary treated effluent to Dredger Cut were also considered and evaluated in some detail. However, the original assumption behind these alternatives was that only a secondary level of treatment would be required during months when contact recreation and agricultural irrigation using water from Dredger Cut were considered unlikely. After extensive discussions with Regional Board staff, it was apparent that long term discharge of any secondary treated effluent to Dredger Cut would not be allowed under the new permit. These alternatives also would have required large amounts of additional land and storage. Therefore these alternatives were dropped from further consideration.

Construction of the Sports Complex does not significantly affect the combinations of alternatives required to meet municipal effluent discharge requirements because the majority of the treated wastewater will still have to be discharged or reused somewhere else. If the Sports Complex covers the cost of tertiary filtration and disinfection for a portion of the flow, it could affect the net costs to the City for some of the combinations of alternatives. This will be addressed later in Section 13—Evaluation of Treatment and Disposal/Reuse Alternative Combinations.

SATISFYING OTHER DISCHARGE REQUIREMENTS

Municipal Effluent – Animal Feed Crop Irrigation

The anticipated and potential future requirements for animal feed crop irrigation can be reliably met with existing treatment processes.

Municipal Effluent – Unrestricted Irrigation

Unrestricted irrigation reuse of municipal effluent for the proposed sports complex or other uses will require tertiary filtration and enhanced disinfection. The reasonable filtration alternatives are:

- Sand filters
- Membrane filters
- Synthetic compressible medium (fuzzy) filters

Disinfection alternatives include:

- Gas chlorination
- Liquid chlorination (hypochlorite)
- Ultraviolet disinfection

These filtration and disinfection alternatives are discussed in greater detail in Sections 11 and 13.

Industrial Wastewater Discharge to Land

Since all industrial wastewater (which is primarily cannery waste) is land applied, the discharge requirements are related to the prevention of nuisance odors and adverse impacts to groundwater. The anticipated discharge requirements can be met with some changes in cropping patterns and assuming no loss of land to the Sports Complex. Additional land and better distribution facilities would improve operations and enable potential future requirements to be more easily satisfied. Additional solids removal prior to land application, using screens or dissolved air flotation, could also reduce the potential for nuisance odors. Improvements to industrial wastewater disposal/ reuse facilities are discussed in greater detail in Section 8.

Biosolids Disposal/Reuse

Current and anticipated biosolids discharge requirements can be satisfied with existing facilities. The loss of land to the Sports Complex could limit flexibility in future biosolids disposal/reuse operations. Improvements in mixing with municipal and industrial effluent and/or distribution improvements would help satisfy potential future discharge requirements for land application of Class B biosolids on City-owned property. These improvements are discussed in Section 9.

SECTION 6. EVALUATION CRITERIA

This section presents the criteria which were developed for the evaluation of treatment and disposal/reuse alternatives. These criteria were developed together with Lodi City staff and the Public Advisory Committee to be representative of a wide spectrum of interests.

ECONOMIC CRITERIA—COST

Life cycle costs are typically one of the most important considerations when comparing alternatives. Life cycle costs include the capital cost of improvements plus the present worth of future operations, maintenance, and other recurring costs. A discount rate of 7 percent for a 20-year period was selected for life cycle cost calculations in the wastewater master plan evaluations.

SUBJECTIVE CRITERIA

General

Subjective criteria are criteria which cannot be easily quantified in terms of direct or indirect costs, yet have a significant effect on the overall success of an alternative. These criteria will be evaluated subjectively for each alternative where they are relevant. The results of the evaluation of subjective criteria will then be used to choose from alternatives which are similar in terms of life cycle costs.

Compliance with Potential Future Discharge Standards

Compliance with current and anticipated discharge standards is mandatory. This requirement has already been used to screen alternatives in Section 5. The ability to satisfy potentially more stringent future standards is a desirable feature for an alternative and was used as a subjective evaluation criterion in this study.

Reliability

Reliability refers to the anticipated reliability of major equipment and processes. Reliability is important not only for minimizing operations and maintenance costs, but for assuring that discharge requirements will not be violated. Processes that have a proven track record are generally considered more reliable than new or experimental treatment methods. Processes or combinations of processes which provide redundant capacity or storage will also contribute to better reliability.

Flexibility

Flexibility refers to the ability to meet future conditions that are now undefined and may change over time. For example, if influent concentrations of key constituents change significantly or if discharge requirements change in an unexpected manner, flexibility refers to how easily those changes could be handled using the proposed facilities or through minor modifications. The ability to increase capacity through simple, inexpensive modifications also indicates a high degree of flexibility. In addition, flexibility refers to how well major equipment or processes would perform during temporary downtime of related equipment or processes.

Ease of Operation and Maintenance

Ease of operation refers both to the operation and maintenance of individual equipment or processes as well as the coordination of multiple related processes. Processes which are conceptually simple, more stable, and/or require less operator intervention will generally be rated higher in terms of ease of operation and maintenance.

Ease of Implementation

Ease of implementation can sometimes be partially quantified in the estimated capital cost of alternatives. However, it is very difficult to estimate the extra management and overhead costs associated with alternatives which are difficult to implement. The ability of a given alternative to be completed in a timely manner and begin operation will be evaluated in regard to potential political constraints, requirements to negotiate agreements, land purchase, construction concerns, difficulties in implementing new technology, and any other similar factors which could affect its successful completion.

Environmental Impacts or Benefits

This criteria includes any significant negative impacts or positive benefits that could result from the implementation of each process alternative. This generally refers to factors beyond those addressed in the anticipated discharge requirements such as wildlife habitat benefits, improvements over background water quality, reductions in dust, *etc.*

Safety

The relative safety of an alternative is related to its potential to cause serious injury in the event of an equipment failure or operational error. Safety applies both to risks to treatment plant staff and to the general public who may be near the treatment plant.

Potential Recreational/Open Space Benefits

This criteria refers to any benefits that could be realized by the public as incidental aspects of project implementation. This could include bird watching features, educational features, community recreational areas, and other benefits.

Aesthetics

Aesthetics include considerations such as visual appeal, odors, noise, and traffic impacts. Aesthetics will be used to apply to new physical facilities so as not to be confused with Recreational/Open Space Benefits.

Secondary Economic Benefits

There are secondary economic benefits or impacts to some of the potential alternatives for effluent disposal/reuse. This could include factors such as additional farm income in the area or additional employment opportunities or taxes from businesses using reclaimed water.

Resource Management Considerations

This criteria refers to the overall best use of resources managed by the wastewater treatment department. This would include the use of fresh water, treated effluent, biosolids, energy, and land.

RELATIVE WEIGHTINGS OF CRITERIA

The relative weightings of the evaluation criteria were discussed in meetings with City staff and the Advisory Panel. The consensus relative weightings developed in these meetings were as shown in Table 6-1.

Table 6-1. Subjective Criteria Weightings

Criteria	Relative Weighting
Compliance with Potential Future Discharge Standards	1.5
Reliability	1.5
Flexibility	1.5
Ease of Operation and Maintenance	1.0
Ease of Implementation	1.0
Environmental Impacts	1.0
Safety	1.0
Potential Recreational/Open Space Benefits	0.5
Aesthetics	0.5
Secondary Economic Benefits	0.5
Resource Management Considerations	0.5

The weightings shown in Table 6-1 will be used in the evaluation of alternatives for later Sections of this Master Plan Report. It should be noted that not all criteria will necessarily be applicable to every set of alternatives. While there is no firm standard for evaluation criteria or the weightings of criteria, these relative weightings are reasonable in comparison to criteria and weightings used for other similar projects.

SECTION 7. WETLANDS TREATMENT

INTRODUCTION

Constructed wetlands offer the potential to provide polishing treatment, effluent reuse, and storage. The purpose of this section is to provide an initial evaluation of the appropriateness of wetlands for the Lodi wastewater system. Hydraulic loading and balancing issues are discussed subsequently in Section 8. The combined treatment and disposal/reuse train alternatives are evaluated in Section 13—Evaluation of Treatment and Disposal/Reuse Process Train Alternatives.

BACKGROUND

Constructed wetlands consist of horizontal flow treatment units with treatment occurring as the wastewater passes slowly through emergent vegetation. Free water surface (FWS) wetlands with water levels ranging from 1 to 2 feet deep are the most common type of constructed wetland and the most appropriate for the City of Lodi. FWS wetlands can be designed for removal of BOD, TSS, nitrogen, heavy metals, or for temperature reduction. FWS wetlands can also serve to dechlorinate an effluent that has been chlorinated for pathogen destruction. A schematic of a FWS constructed wetlands that includes open water zones is shown in Figure 7-1.

The advantages of constructed wetlands are their low cost of operation, their ability to accept a range of peak flows and variations in loadings, and their attribute as wildlife habitat. Constructed wetlands can provide relatively high water use, nitrate-nitrogen removal treatment, heavy metals removal treatment, and temporary water storage. The water level in wetlands can fluctuate from 0.5 to 3 feet to provide storage capacity.

The primary disadvantages of wetlands are relatively high initial cost and no crop revenue. Other disadvantages include limited nitrification potential and the need to manage the propagation of mosquitoes. The effluent from constructed wetlands is often low in dissolved oxygen as a result of the reducing conditions in the wetlands.

AREA REQUIREMENTS FOR WETLANDS

Each of the treatment objectives discussed in Section 5 – metals removal, temperature reduction, and nitrate reduction – require different detention times. Metals removal requires the longest detention time, which determines the land area required. Based on a flow of 8.5 mgd, the area required is 130 acres for roughly a 50 percent removal of zinc. For nitrate reduction, as preliminary treatment prior to irrigation or recharge, an area of 65 acres would be sufficient. Area for berms, roads, and setbacks should be added to the field area calculated.

WETLANDS PERFORMANCE CHARACTERISTICS

Water Use

Constructed wetlands consist of a combination of open water (20 to 30 percent) and dense emergent vegetation (70 to 80 percent). As a result, the water use or consumption of a

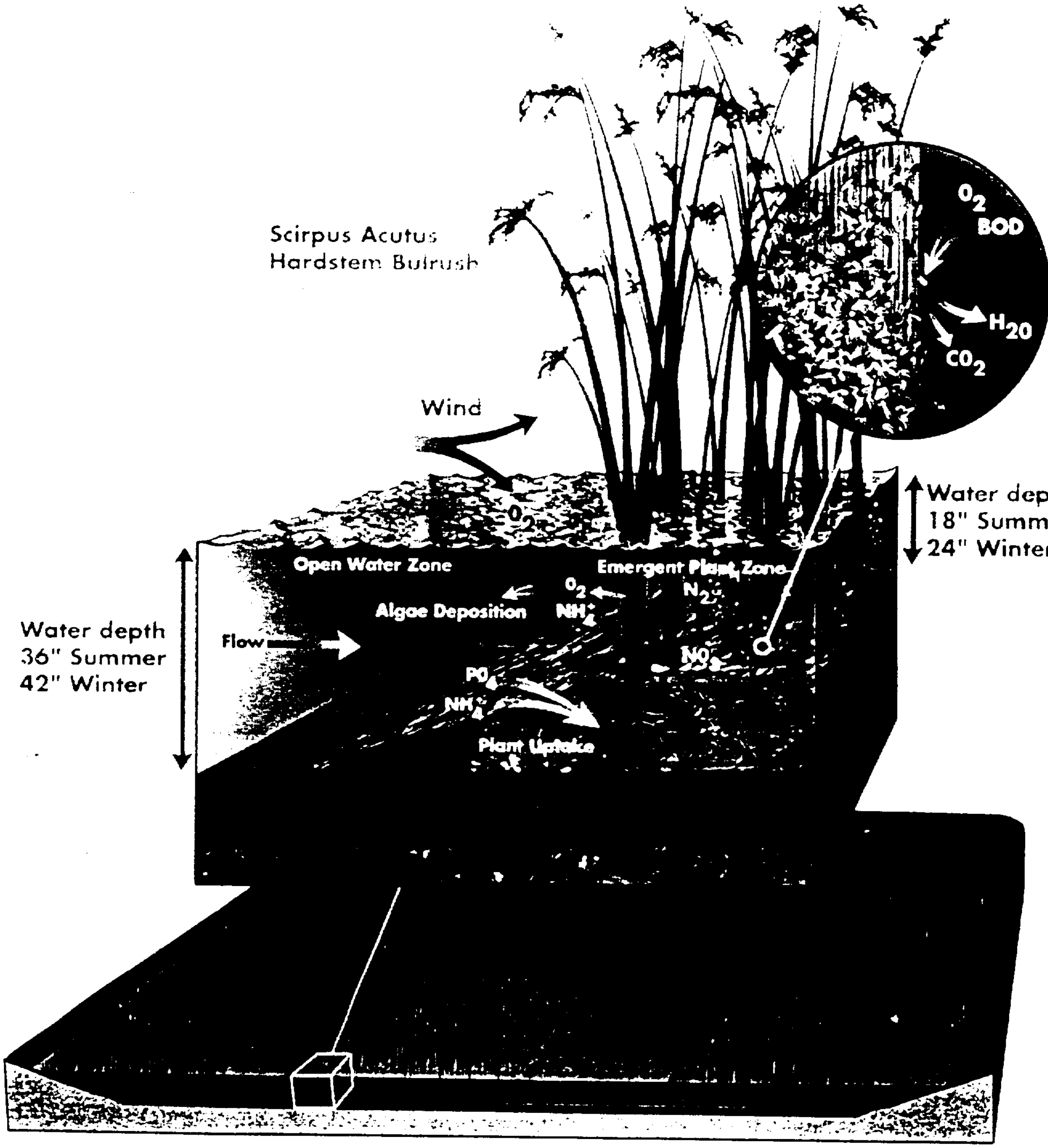


Figure 7-1
Constructed Wetlands Schematic

constructed wetlands can be estimated by using an open water evaporation rate. Assuming a net evaporation of 3 ft/yr, the 130-acre constructed wetland would use 390 acre-ft/yr.

Storage Capacity

Constructed wetlands typically operate at a depth of 1 to 1.5 feet. Shallower depths require more land for the same detention time and deeper depths will affect the emergent vegetation. During the winter months (say November through March) or for shorter durations other times, the bulrush can tolerate periodic inundations of up to 2.5 to 3 ft deep. The dormant season storage can then be expected to be 1.5 acre-ft/acre of wetlands.

Temperature Reduction

Wetlands can reduce the temperature of the wastewater by a combination of shading and atmospheric cooling. The effluent cooling effects are greatest in the winter when the difference between average ambient air temperature and the wastewater temperature is the greatest. At Sacramento County's Demonstration Wetlands, the winter temperature loss through the wetlands averaged 10°F. During summer months the difference dropped to 2.2°F. An initial gradient of temperature loss of –3°F per 100 ft was reported for the pilot wetlands configuration of the Sacramento Regional Wastewater Treatment Plant. For the 5-day detention time wetland in Lodi, a reduction of 8 to 10°F can be expected in the winter months and a 2 to 3°F drop in the summer temperature can be expected. This would allow the temperature differential requirements to be met for discharge to Dredger Cut.

Nitrate Removal

Nitrate removal by denitrification can be achieved through constructed wetlands with detention times of from 2 to 3 days. The nitrate levels created by in-plant nitrification can be reduced in the wetlands because of the anoxic conditions prevalent in free water surface constructed wetlands.

Metals Removal

Metals are removed in constructed wetlands by a combination of plant uptake and burial in the root zone, precipitation, adsorption, and complexation[1]. The removal of metals in free water surface wetlands was studied over five years at the Sacramento Regional CSD Demonstration Constructed Wetlands project in Elk Grove. The removals of six key metals are summarized in Table 7-1.

Table 7-1. Removal of Metals in Constructed Wetlands[2]

Metal	Influent, ppb	Effluent, ppb	Percent Removal
Cadmium	0.25	0.03	87
Copper	7.44	3.17	57
Mercury	10.77 ppt	4.01 ppt	63
Lead	1.14	0.23	80
Nickel	5.80	6.84	-18
Zinc	35.8	6.74	81

The removals presented were obtained using secondary effluent with a detention time of 7 to 10 days. The only major metal that resisted removal was nickel, which was 80 percent in the dissolved form and not readily removable.

BOD and Suspended Solids Polishing

BOD and TSS removal in constructed wetlands can be achieved in relatively short detention times. Secondary effluent can be polished down to the 10 to 15 mg/L level. Plant upsets can be absorbed by the wetlands without adverse effects on the wetlands effluent.

Pathogen Removal

Typical removal of influent pathogens and indicator organisms in constructed wetlands is one to two logs. Die-off and predation occur in wetlands at rates similar to facultative ponds. Fecal coliform removals at Arcata ranged from 80 percent in the winter to 95 percent in the summer. At Listowel and Iselin, 99.9 percent removal of fecal coliforms were reported[1]. Wildlife in wetlands contributes a baseline level of coliform bacteria which is not easily distinguished from human pathogens in the standard tests. So although wetlands remove human pathogens from wastewater, this removal is not easily measured or given numeric credit by regulatory authorities. Disinfection prior to or after wetlands is usually required to meet California standards for discharge or reuse. If disinfection is prior to the wetlands, the point of compliance for disinfection standards can be applied at the outlet of the disinfection facilities.

Dechlorination

Established wetlands have substantial amounts of available carbon. Free chlorine residual is readily removed from the water column by reaction with available carbon. Wetlands can serve as an effective buffer to assure complete dechlorination after conventional dechlorination with sulfur dioxide or sodium bisulfite. This would allow conventional dechlorination to be performed without worrying about maintaining an excess of dechlorinating agent and with a greatly reduced risk of receiving water impacts in the event of equipment or control system failure.

WETLANDS SITE ALTERNATIVES

Wetlands alternatives were developed for combined polishing treatment, effluent disposal/reuse, and storage. The wetlands site alternatives are:

- Use of the existing DWR wetlands
- Construction of new wetlands north or east of the City's current irrigation area
- Constructed wetlands immediately west of the treatment plant
- Outfall wetlands constructed south of Dredger Cut on the Rio Blanco site

Use of Existing DWR Wetlands

An existing wetlands site northwest of the treatment plant has been considered for potential use as a wastewater treatment option. The site is parallel to the High Line Canal and the alignment of the Peripheral Canal and Highway I-5. The California Department of Water Resources (DWR) controls the land. Dave Brown from DWR indicated that the site is earmarked for either water conveyance (canal) or for mitigation. Neither use seems compatible with Lodi's wastewater treatment goals. The wetlands are considered waters of the state and therefore would require that effluent water quality be similar to that in a surface water discharge.

Constructed Wetlands North/East of the City's Current Irrigation

The farmland to the east of the DWR wetlands could be converted into a constructed wetland for wastewater treatment. The benefits of this site would be compatibility (sharing of edges) with the existing DWR wetlands, treatment capability for denitrification, temperature reduction, and metals reduction. The site would be converted to a free water surface constructed wetland by creating berms, open water areas and deep water (>3 ft) areas. For treatment of 8.5 mgd, the net wetland treatment requirement would be approximately 130 acres assuming that zinc removal is the most restrictive factor. Treated effluent from the wetlands could be discharged or used for irrigation or aquifer recharge/recovery.

Constructed Wetlands Immediately West of the Treatment Plant

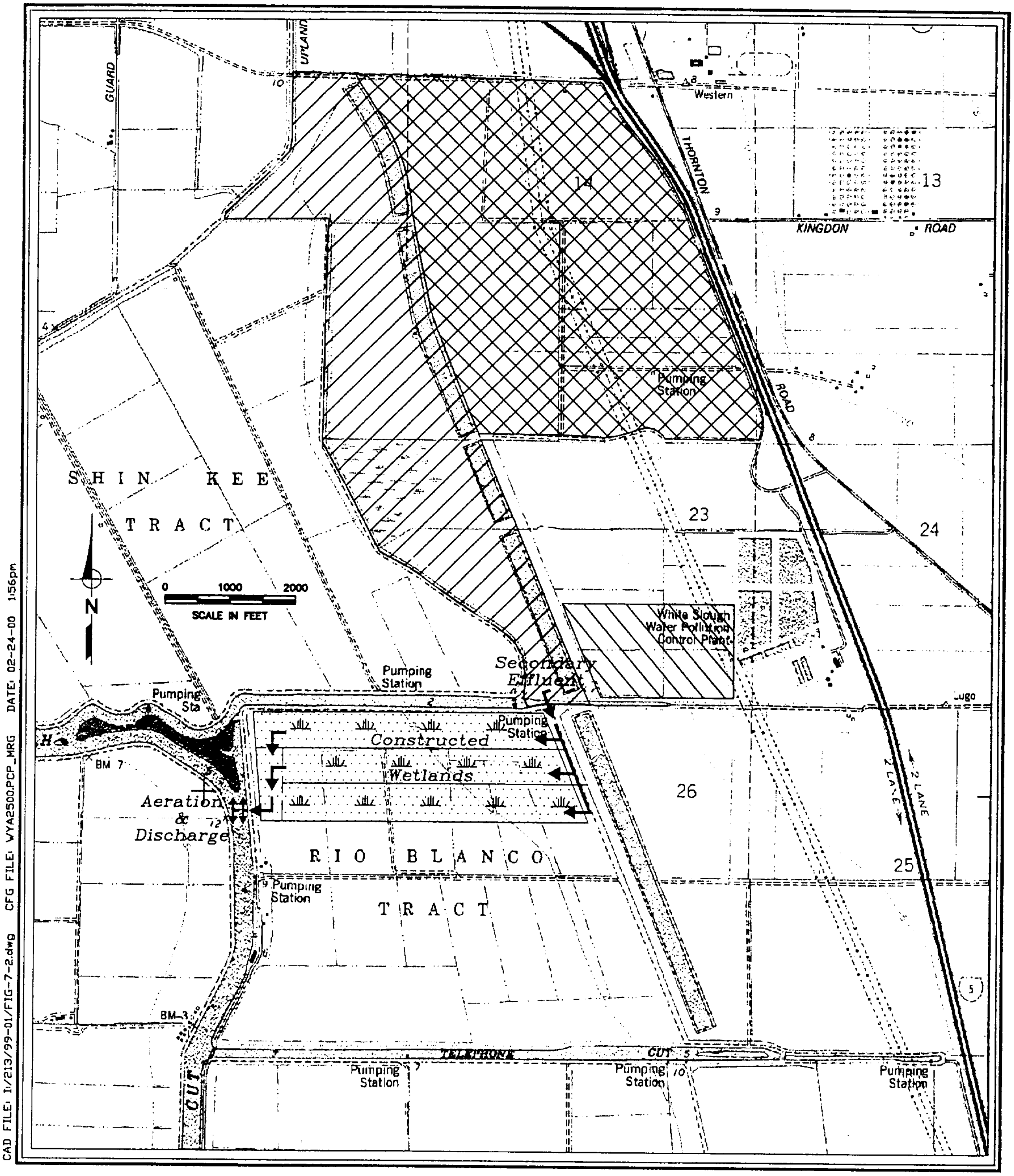
A small (50 – 100 acres) wetland could be constructed immediately west of the treatment plant on land currently irrigated with reclaimed water. This location would be easy to accommodate hydraulically because it could be gravity-fed from the treatment plant. Effluent from a constructed wetland at this location could be easily returned to the storage ponds for subsequent conveyance to percolation basins or fields. A small wetlands would still provide adequate denitrification, but would provide proportionately less zinc removal and temperature equalization than a 130-acre wetland.

Outfall Wetlands Constructed South Of Dredger Cut On The Rio Blanco Site

The use of constructed wetlands at this site would allow a surface discharge into Bishop cut without the cost of an extended outfall. The influent to the wetlands would flow into the eastern edge of the wetlands, as shown in Figure 7-2, and treatment would occur as the water moved westward through the wetlands, ending up at a lift station next to Bishop Cut. Reaeration would be provided by pumping the effluent over a cascade aeration system.

COST OF CONSTRUCTED WETLANDS

Typical costs per acre of FWS constructed wetlands can range from $20,000 to $40,000/acre depending on the site work, need for liners, interior berm spacing, and planting density. The three sites are relatively level so that site work would not be extraordinary. With the application of secondary effluent, there should not be a need for artificial liners. To minimize the planting costs it is recommended that the wetlands be planted in strips perpendicular to the flow path, with unplanted and open water zones in between the planted areas. The planting would consist of cattail seeding and a moderate amount of transplanted bulrush. The shallow unplanted areas will be readily colonized by the bulrush that are transplanted. For the Rio Blanco site the cost should be about $20,000 to $25,000/acre without land costs.



White Slough Wildlife Area (DWR)

Northern Alternative Wetlands Site

Rio Blanco Alternative Wetlands Sites

Immediate West Alternative Wetlands Site

Figure 7-2

City of Lodi
Wastewater Master Plan

Wetlands Site Alternatives

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Special Wetlands Funding Sources

Special funding sources exist for the construction of wetlands to increase the acreage of habitat for wildlife. Agencies and organizations that have funding programs for the development of wetlands include the Natural Resources Conservation Service, U.S. Fish and Wildlife Service, the State Department of Fish and Game, and CALFED. Ducks Unlimited supports many wetlands development projects, but usually is fully consumed with large fresh-water wetlands projects. EPA has some wetlands grants available through the Association of Metropolitan Water Agencies (AMWA). Drinking water suppliers are eligible to receive the grants, which are to be used to establish new wetlands programs or refine existing ones. The USF&WS has a partners program that supports wetlands enhancement and development.

ENVIRONMENTAL BENEFITS OF WETLANDS

The environmental and community benefits that come with constructed wetlands include wildlife habitat, recreational benefits from bird watching and hiking, and educational benefits to school groups and community groups that can visit the wetlands. Wildlife habitat includes that for song birds, ducks, and geese. Audubon Societies are often the strongest supporters for the development of constructed wetlands.

WETLANDS MANAGEMENT ISSUES

To manage a constructed wetlands requires attention to the flow path (avoiding short-circuiting), attention to the vegetation and berms (controlling any burrowing animals or pests), and control of mosquito production. Mosquito control requires multiple management techniques and frequent sampling during the April through October time frame. Management techniques developed in the 5-year demonstration wetlands at Sacramento County include introduction of mosquitofish, the use of BTI or other bacterial parasites, and encouragement of natural predators. Harvesting of vegetation is not generally required except as needed to allow mosquito control practices.

CONCLUSIONS

Constructed wetlands offer potentially significant benefits in the overall treatment and disposal/reuse system. The Rio Blanco site is the most promising in terms of benefits versus cost if treated effluent is to be discharged to Bishop Cut or Dredger Cut and 50 percent zinc removed is a design criteria. For the Land Discharge alternative, a wetland located immediately west or north of the existing treatment plant would provide denitrification, disposal, storage, and environmental enhancement benefits closer to the best irrigation reuse and percolation disposal areas. Combined treatment and disposal train alternatives which include wetlands alternatives are evaluated in Section 13. If the need for zinc removal is reduced through source control or tertiary treatment, a smaller wetland (65 acres) immediately west of the treatment plant or on the Rio Blanco site would be preferred.

REFERENCE

[1] Crites, R.W. and G. Tchobanoglous. *Small and Decentralized Wastewater Management Systems*. McGraw-Hill. New York. 1998.

[2] Nolte Associates, *Executive Summary, Sacramento Constructed Wetlands Demonstration Project, Five-Year Summary Report, 1994-1998*. January 2000.

SECTION 8. INDUSTRIAL WASTEWATER TREATMENT AND DISPOSAL/REUSE

INDUSTRIAL SOURCES

Some industries discharge to a segregated industrial wastewater line that enters the White Slough wastewater treatment facility adjacent to the domestic line. This wastewater is then pumped to the distribution system for direct application to farmlands. Approximately 95 percent of the industrial wastewater flow comes from Pacific Coast Producers (PCP). The remainder comes from M&R Packing and several small manufacturers. PCP processes apricots, peaches, and tomatoes for a variety of canned products. The processing season begins in early Summer with apricots, then peaches and tomatoes, with the peak flows generally occurring in August of each year. There is little or no wastewater flow during the remainder of the year.

During 1998 and 1999, flow rates from PCP were higher than in past years. The flows during the last three years are shown on Figure 8-1 (identical to Figure 3-10). The 1999 increase was due to an expansion of tomato processing facilities and the corresponding increase in tomato processing wastewater flow. PCP plans to increase its recycled flows prior to the 2000 season in order to obtain a one million gallons per day (1 Mgd) reduction in wastewater flows.

PACIFIC COAST PRODUCERS WASTEWATER SOURCES AND PRETREATMENT

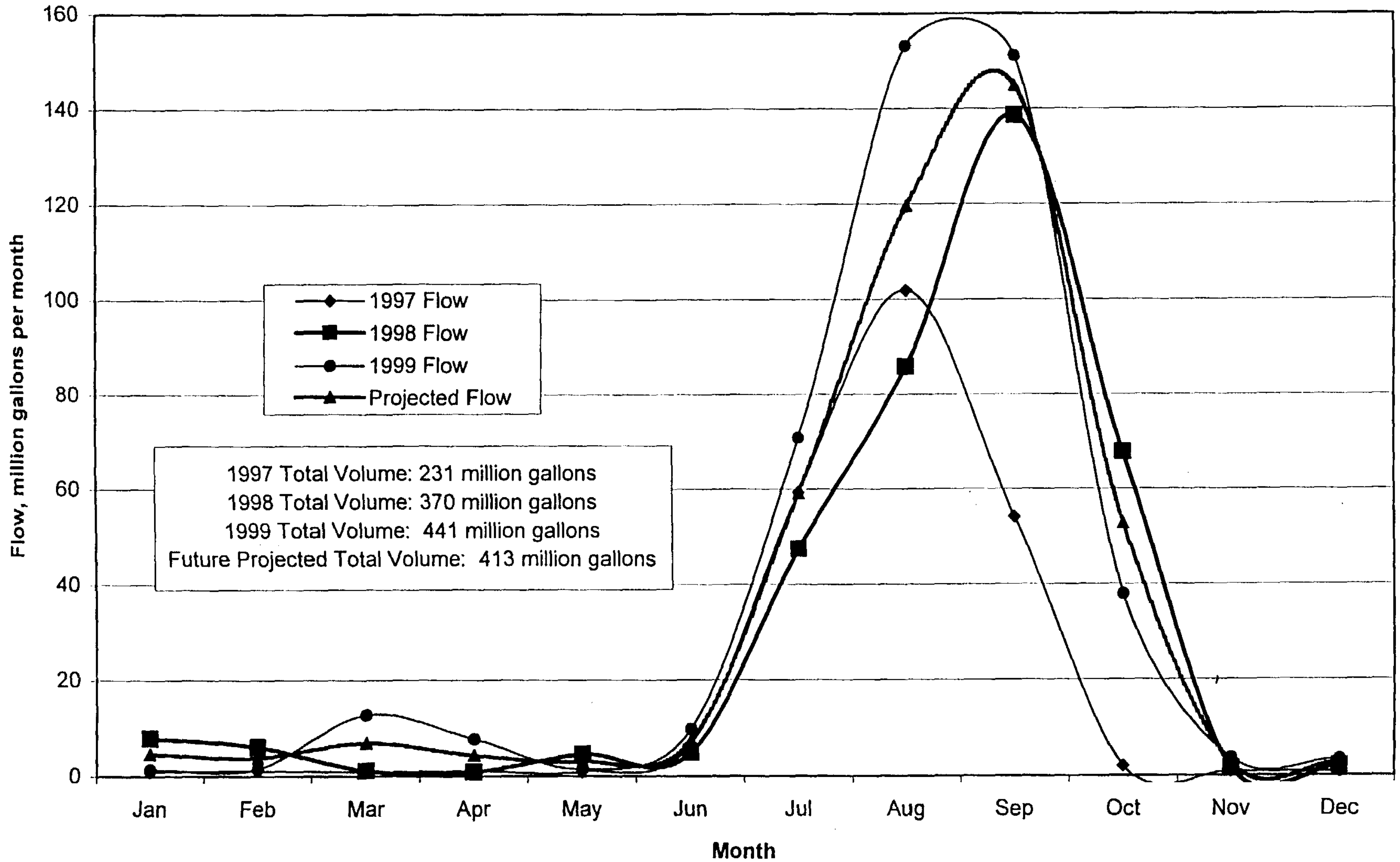
Pacific Coast Producers wastewater comes from the following sources:

1. Washwater from washing and transporting incoming fruit and vegetables
1. Boiler blow down wastewater
2. Wastewater from caustic peeling of apricots, peaches, and tomatoes
3. General factory washdown water

The washwater from the incoming product area is treated in mud removal chambers to settle and remove some waste material. The collected mud is placed in a container for off-site disposal. This wastewater source is high in fine suspended solids. Boiler blow down wastewater is low in suspended solids and BOD but can be high in total dissolved solids (TDS). Wastewater from the peeling operation varies depending on the type of peeling operation employed. Apricots and peaches are peeled using a caustic peeling system. Tomatoes are peeled using either a caustic peeler or steam peeler. Both peeling operations produce a wastewater high in suspended solids and BOD. The caustic peeling operation produces a wastewater that also has high concentrations of both TDS and sodium.

The average TDS, total fixed dissolved solids (TFDS), and pH values for the 1998 season were 1,369 mg/L, 717 mg/L, and 9.8, respectively. The mineral salinity as measured by TFDS is moderately elevated, and the pH is relatively high. TFDS and sodium are undesirable in land-based wastewater treatment systems where crops are grown. Sodium will cause infiltration rates in clay soils to become so low that water will not penetrate the soil surface. High TFDS levels in soils and

FIGURE 8-1. 1997 - 1999 AND PROJECTED INDUSTRIAL WASTEWATER FLOWS



water will cause crop damage. No problems associated with elevated levels of TFDS and sodium have been observed to date. TDS is not an accurate measure of salinity for food processing wastewater because it includes organic compounds which break down when treated or applied to land.

Pretreatment has historically consisted of parabolic screens (0.020-inch openings) to remove solids. The parabolic screens were enlarged this past summer. During the 1999 processing period, abnormally large quantities of solids were observed in the wastewater from the tomato processing. It was later discovered that the new screen had slipped in its frame leaving a gap where solids discharged directly into the effluent channel. This problem was corrected by fixing the screen.

Solids coming off the screens enter a conveyor system that discharges the solids into a container for off-site disposal. With the plant expansion this past summer the conveyor system was sometimes overloaded and solids fell into the effluent channel, thus defeating the purpose of the screening system.

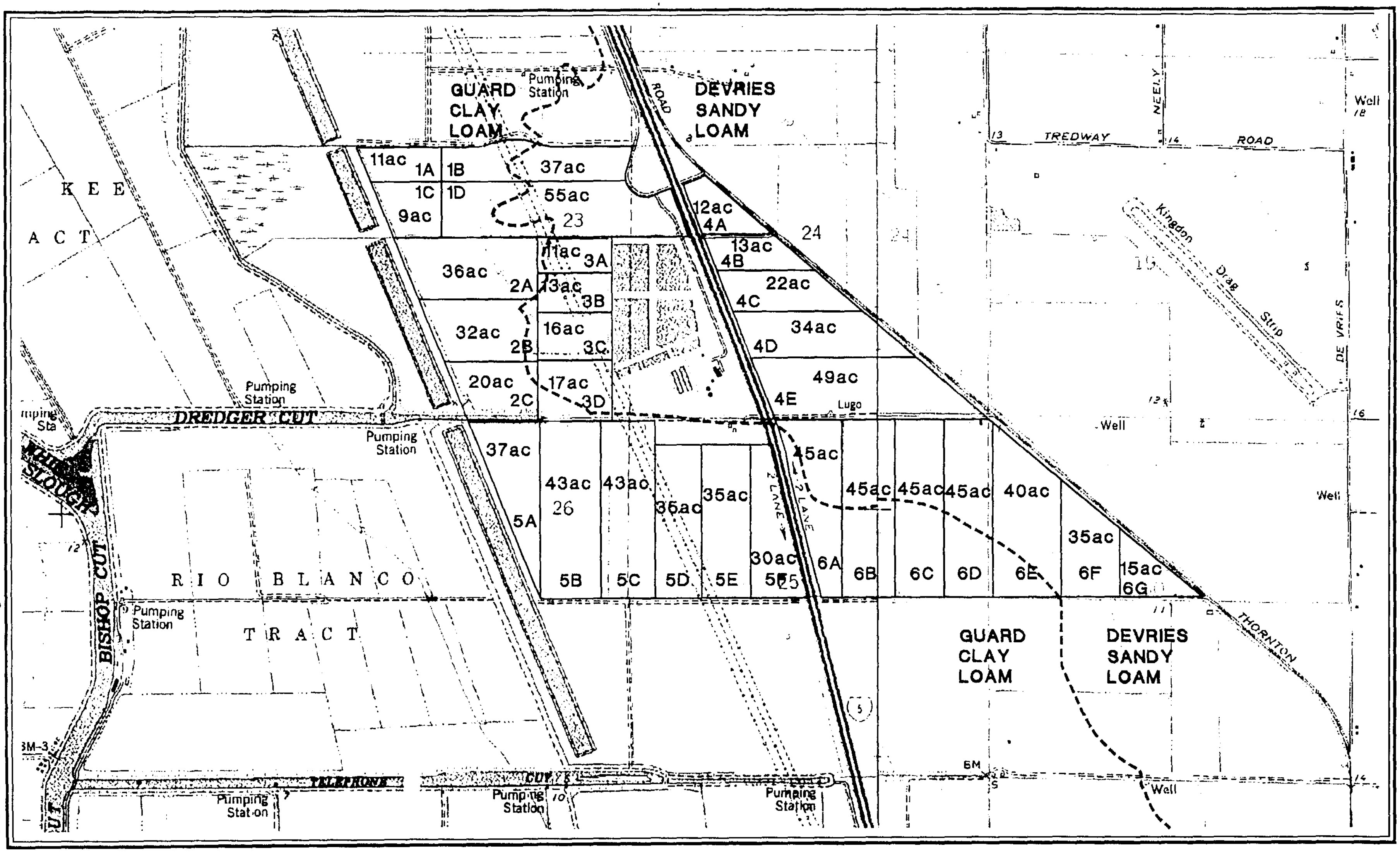
During the processing season (1999), PCP installed pH adjustment and recycling systems to handle wastewater from the caustic peeler for the tomato processing. These systems convert the peeler wastewater to a neutral water supply high in salt that is then used in canned tomatoes as a salt source. Installed late in the 1999 season, the operation of this system still requires some adjustment. Full utilization of this system will greatly reduce TFDS and sodium discharged to the City.

The water supply/recycling system was modified to recycle approximately 1 Mgd of wastewater for washing incoming fruit and tomatoes. Fresh water and other recycle streams are all routed to a sump that has several pumps that deliver water to a variety of destinations. The modification made to recycle the additional 1 Mgd of wastewater did not work and has not been used. PCP is re-designing a new system to achieve the recycling goal.

EXISTING INDUSTRIAL WASTEWATER TREATMENT SYSTEM AT WHITE SLOUGH

The treatment and disposal system at the White Slough facility is the simplest, most economical system that could be employed to treat and dispose of cannery wastewater. The reason for this is that there are no mechanical treatment processes to operate and maintain. Cannery wastewater is not treated at White Slough other than by coarse screening to remove large solids. The wastewater is directly applied to the farmland owned by the City surrounding the White Slough facility. The individual field designations and sizes are shown on Figure 8-2.

Alfalfa and feed corn are grown during the summer months with oats or small grains grown on the corn ground during the winter months. If not controlled, excessively high solids concentrations or high TDS levels will create problems for a system of this type. The high solids will results in odors in the fields and/or crop damage where the irrigation water first enters the fields. Effective screening systems at the cannery will be adequate to prevent this from occurring. As pointed out above, there were excessive solids found entering the system from PCP during the summer of 1999. The excessive solids were a result of a misaligned parabolic screen and spillage of solids from the conveyor system removing the solids from the



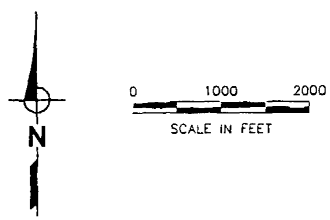


Figure 8-2

City of Lodi
Wastewater Master Plan

Existing City-Owned Fields

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CAD FILE: I:/213/99-01/FIG-8-2.dwg CFG FILE: WYA2500.PCP_MRG DATE: 09-18-00 3:16pm screening system. The miss-alignment was repaired. The conveyor system still needs modification. The problem can be solved by enlarging the conveyor and by covering the effluent channel so that spilled solids are routed to the pump sump rather than to the effluent channel.

During the summer of 1999 alfalfa production was impacted due to too much water in the root zone causing root rot and other root damage. The damage was caused by extended periods of an anaerobic soil condition. The anaerobic condition was caused by excessive water in the root zone (alfalfa roots can be as deep as 6 feet and are very sensitive to extended periods of excessive moisture). This problem was due to a number of factors, one of which was over-irrigation when the farmers had to add treated wastewater to dilute the cannery wastewater. With the improved solids removal at the PCP facility, the wastewater should not require as much dilution, thus reducing the total amount of water applied to the crops.

Direct application of cannery wastewater is a very efficient and cost effective means of treatment and disposal. Misapplication or over-application can result in development of odors and crop damage. To prevent these developments, application rates are set based on BOD loading. The loading rate is set to match soil type. The BOD limitation is based on the rate that oxygen can be transferred to satisfy the demand of the aerobic bacteria in the soil. If the oxygen transfer is limited, anaerobic conditions may develop leading to odors.

The following oxygen diffusion model estimates the maximum oxygen diffusion rate to the soil for one day after waste application (Overcash and Pal, 1979). This equation can be used to calculate allowable BOD loading rates for land application of wastewater.

$$N_{O_2} = 2(C_{O_2} - C_P)[DT/\pi]^{1/2}$$

$$= 80\ \text{g/m}^2\text{/d}$$

$$= 714\ \text{lbs/ac/d}$$

where:

N_{O_2} = the flux of oxygen crossing the soil surface, ML

C_{O_2} = vapor phase O_2 concentration above the soil surface = 300 ppm

C_p = vapor phase O_2 concentration required in the soil to prevent adverse yields or root growth = 140 ppm

T = time over which diffusion is occurring, 1 day

D = effective diffusion coefficient

D = $0.6(S)(D_{O_2})$

where:

S = air filled soil pore volume at field capacity

D_{O_2} = oxygen diffusivity in air = $1.6\text{m}^2\text{/d}$

There are two predominant soil types on the City-owned land surrounding the wastewater treatment facility. They are Devries sandy loam and Guard clay loam. Estimated allowable organic loading rates for these two soils calculated using the above equation are shown in Table 8-1.

Table 8-1. Allowable BOD Loading Rates

Soil Type	Permeability	Net Pore Space, S	N_{O_2}, lb/ac/d	Reaeration Time Factor	Allowable Loading, lbs BOD/ac/day
Devries Sandy Loam	Moderately rapid	0.27	825	.8	560
Guard Clay Loam	Moderately slow	0.27	825	.6	420

These loading rates are based on the assumption that the application is made with a number of days of rest in between applications. For tilled soils the application should be made with a minimum of three days rest between applications, and land should be disced after the soil has dried adequately to support tractor and tillage implements. Applications on the alfalfa or corn fields should be made with a 10-14 day rotation as is practiced in conventional agricultural operations. The allowable application rates determined by the above formula are the average amounts applied over the entire period. Instantaneous daily application rates will be much greater. Experience with other sites has shown a maximum 2,000 lb/ac/day on one day will not create odor conditions as long as the average value does not exceed the recommended one.

It is also important to consider the effect of the BOD loading rates in conjunction with the hydraulic loading rates, especially for crops which are somewhat sensitive to suffocation such as alfalfa. This was evaluated during the 1999 season when the hydraulic application rate of combined industrial and domestic effluent exceeded the hydraulic capacity of the irrigated acres, resulting in some crop damage and odors. The loading rates in Table 8-1 are only appropriate for crops which are irrigated at rates which meet, not exceed, evapotranspiration requirements.

With both soils the Natural Resource Conservation Service recommends in its soil survey that occasional soil ripping and/or drainage enhancement projects may be necessary to maintain adequate soil conditions for the system to work successfully. Areas mapped as the Devries soil can have hardpan at 15 inches to as much as 80 inches below the surface. The hardpan can be broken up mechanically by deep ripping between crop rotations. Re-leveling fields to prevent ponding will also minimize formation of cemented soils with the Guard series soils.

ACREAGE REQUIRED

BOD Loading

In order for a direct land application system to work, the loading rates must be controlled based on the previously presented formula. The acreage required is determined by the acceptable loading rate. At the 1999 peak weekly flow, 142,000 lb/day of BOD from the cannery was applied to 790 acres (net) farmland. This equates to a daily application rate of 180 pounds

BOD/acre/day. Using the lower number presented in Table 8-1 (420 lbs. BOD/ac/day), the ranch can accept more cannery wastewater. At 420 pounds BOD/ac/day, 338 acres would have been the minimum recommended area for disposing of the 1999 cannery flows.

Using the projected cannery loading (from Section 3) of 2,700,000 pounds BOD/month and a maximum loading of 420 pounds BOD/ac/day, the theoretical minimum area recommended based on BOD loading criteria would be 210 acres. Because of crop rotation, harvest operations, and irrigation scheduling considerations, a realistic minimum area based on BOD loading would be about double the theoretical minimum, or 420 acres.

Hydraulic Loading

The maximum hydraulic loading for the 790 net acres available for irrigation is controlled by a combination of industrial effluent flows and treated domestic effluent flows. The cannery flows generally peak in mid- to late-August or early September. The projected industrial flows were shown in Figure 8-1. Assuming an evapotranspiration rate of 6.4 inches for the month of August, an irrigation efficiency of 75 percent, and a leaching fraction of 0.10, a conventional irrigation application rate would be 9.5 inches for the month. Applying the industrial effluent at this conventional application rate would require a minimum of 600 acres of crops. Therefore hydraulic loading is more limiting than BOD loading for the industrial effluent.

Additional irrigated area is required for disposal/reuse of the domestic municipal effluent. Less land can be used if soil and crop conditions allow irrigation in excess of conventional rates. The risks of adverse crop impacts and odors increase for irrigation at higher rates. Detailed water balances which include industrial effluent and domestic effluent for the various process train alternatives are shown in Section 10.

ALTERNATIVE IRRIGATION METHODS

Currently, irrigation is accomplished using furrows for corn and graded border check systems for alfalfa and small grains. The disadvantages of the furrow irrigation is mediocre distribution of solids and wastewater. The disadvantage of graded border check irrigation is poor solids distribution because of filtration and interception of solids by the crop stems. Poor solids distribution can cause an increase in effective organic loading at the upper end of the graded border check and can significantly reduce soil reaeration by blinding soil pores. This can cause crop damage at the upper end of the field.

One alternative would be to use sprinklers for irrigation. Sprinklers would distribute wastewater moderately better than surface irrigation and would distribute solids substantially better than surface irrigation. However, this option would be very costly to install (approximately $400 to $2,000 per acre) and would require booster pumps to generate enough pressure for the system to work. Sprinklers have the additional disadvantages of clogging and spreading aerosols, which in turn could increase odors from the application site. The benefits from sprinkler irrigation are not sufficient to warrant the costs and other disadvantages at this time.

Other alternatives would be to change the surface irrigation system employed. One possible modification would be conversion to a level basin system. This would provide a system with no tailwater to pump. An advantage of this type of system is that flows are generally much higher during irrigation, which would create greater velocities and spread solids further out into the fields. The major disadvantage of conversion to a level basin system is the extensive regrading to create smaller dead-level fields from the larger sloped fields which currently exist.

A second variation for surface irrigation would be to install "surge" systems. These systems use gated pipe and valves to send the water down the furrows in pulses typically from 10 to 30 minutes apart. This type of system would also push solids further into the field by providing a greater instantaneous flow rate during application. Surge systems would distribute water and solids more uniformly. Surge systems would require the installation of a new network of underground supply pipelines to the fields along with the purchase of the gated pipe, surge valves, and controllers. The cost of a complete surge system would be on the order of $300 to $500 per acre. Surge systems are generally not used with graded border checks.

TREATMENT ALTERNATIVES

The direct land application alternative (as presently practiced) requires no treatment at the White Slough facility. In order for this process to work effectively without producing odors, effective solids removal is necessary by the industrial discharger prior to discharge. All the alternatives to direct land application include treatment for reduction of BOD and then land application or surface water discharge. Treatment could be provided with a fixed film reactor and/or an aerated pond system. Either alternative would require significant capital investment and power cost. Because adequate land is available based on BOD loading rates, conventional treatment of food processing wastewater is not economically justifiable.

PRETREATMENT ALTERNATIVES AT PCP

Completion of the projects mentioned previously for the PCP cannery will help reduce both suspended solids and TDS entering the sewer to White Slough. Assuming continuation of the existing direct land application of the cannery wastewater, efforts should be made to reduce suspended solids. Possible alternatives for a greater reduction in solids would include installation of finer screens or installation of a dissolved air flotation system to remove colloidal solids.

Rotary Drum Screens

The screens in use at the PCP cannery are relatively fine compared to most static screens. An incremental improvement in solids removal performance could be obtained through the use of rotary drum screens. Rotary drum screens tend to require less manual cleaning than static screens and have openings as small as 0.010 inches. The amount of additional solids captured by rotary drum screens with 0.010 inch openings would not justify the cost of switching from static screens with 0.020 inch openings.

Gravity Disk Screens

Gravity disk screens work on the filter precoat principal in which captured solids provide the filter mat for removing additional solids. Gravity disk screens provide very efficient solids removal down to particle sizes much smaller than are achievable with rotary drum screens.

However, the flow capacity of individual gravity disk screen units is relatively low, and the resulting costs are much higher than for static or rotary drum screens. Gravity disk screens are usually employed on food processing waste streams where the recovered solids have a high market value. Therefore, gravity disk screens are not a recommended alternative unless PCP can derive a high value from its recovered solids or if land for reclamation/disposal becomes very limiting.

Dissolved Air Flotation

Dissolved air flotation is commonly used to remove solids from food processing wastewater. It removes approximately 70 to 90 percent of suspended solids, depending upon polymer usage. Dissolved air flotation is particularly attractive when the food processor is discharging to a municipal system or to a land treatment system with limited solids handling capacity.

A 50-foot circular dissolved air flotation tank would be required to treat the projected 4.5 Mgd peak cannery flow rate. A gravity belt filter or belt filter press would probably also be required to dewater the solids prior to disposal or reuse. The capital cost of new dissolved air flotation and solids dewatering facilities at PCP is estimated to be approximately $3 million including polymer feed, support facilities, engineering, and contingency. This would not include redundancy for either the dissolved air flotation tank or dewatering facilities. This also does not include the present worth of operational costs for disposal of dewatered solids.

The main benefits to the City of Lodi from dissolved air flotation at PCP would be moderately improved wastewater infiltration rates and a lower potential for odors from the irrigated fields. These benefits are not sufficient to justify the costs of installing dissolved air flotation at PCP at this time.

SUMMARY AND RECOMMENDATIONS

The existing direct land application of industrial wastewater should be continued along with increased tracking of wastewater application at the White Slough Facility. Increased tracking should include recording of actual fields receiving wastewater so that the hydraulic and BOD loading rates are known for each day. Industrial wastewater should be preferentially applied by furrow irrigation to annual crops rather than by border irrigation.

Improvements recommended for implementation at the PCP cannery include better removal of solids from the wastewater at PCP through redesigned solids conveyance systems and increased recycling to reduce BOD loading to farmland. For long-term viability of the land treatment system, the PCP facility should continue to develop means to minimize sodium loadings in effluent.

SECTION 9. BIOSOLIDS DISPOSAL/REUSE ALTERNATIVES

BACKGROUND

Currently, biosolids are anaerobically digested and discharged into a concrete-lined lagoon. During the summer growing season, the suspended biosolids are pumped from the lagoon at a solids content of about 2 percent and discharged to the fields through the surface irrigation system. Biosolids are sometimes applied to fields before planting in late Spring or after harvest in the Fall. The biosolids lagoon is decanted back to the storage ponds. The supernatant from the lagoon is combined with treated effluent for irrigation. Biosolids from this facility have consistently met the requirements of the Federal Regulations (40CFR503).

The surface irrigation methods used for biosolids application to fields include furrow irrigation and border check (graded basin) irrigation. The furrows typically have been constructed with multiple 90-degree turns rather than straight. The purpose of the turns is to reduce tailwater runoff.

The ability of this system to distribute solids and nitrogen has been questioned by the Regional Water Quality Control Board staff. Even though the solids are suspended in 98 percent water, it is likely that some solids are settling out near the head end of the field rather than being distributed throughout the length of the furrows, graded basins, or fallow fields. The same unequal distribution of the non-soluble portion of organic nitrogen is also likely. This phenomenon occurs with either furrow irrigation of row crops or border irrigation of alfalfa, although it is more pronounced with the border irrigation of alfalfa.

During the summer months, applications are limited to the corn crop. This is due to the 30-day limitation of no harvesting after the biosolids application to alfalfa. Late in the season, when alfalfa harvest is completed, applications are made to the alfalfa fields. This 30-day limitation would be applicable even if alternative methods were adopted for biosolids distribution.

According to plant staff, the land applied biosolids can be a brief source of minor odors during initial drying. Better distribution of solids would minimize the potential for odors.

BIOSOLIDS LOADINGS

Recent and Projected Loadings

The biosolids production and nitrogen content values for recent years and for projected Year 2020 are shown in Table 9-1. Organic nitrogen compounds in biosolids must be first mineralized to ammonia by natural biochemical processes before the nitrogen becomes available for plant uptake or bacterial conversion to nitrate. Total available nitrogen is therefore defined as the total amount of ammonia and nitrate nitrogen. The total available nitrogen was determined by assuming that 50 percent of the ammonia is lost due to volatilization after application and that mineralization rates for organic nitrogen for the first ten years are as per the EPA Process Design Manual[1]. There can be some gradual mineralization after ten years, but it is usually considered to

Table 9-1. Biosolids Production and Nitrogen Content

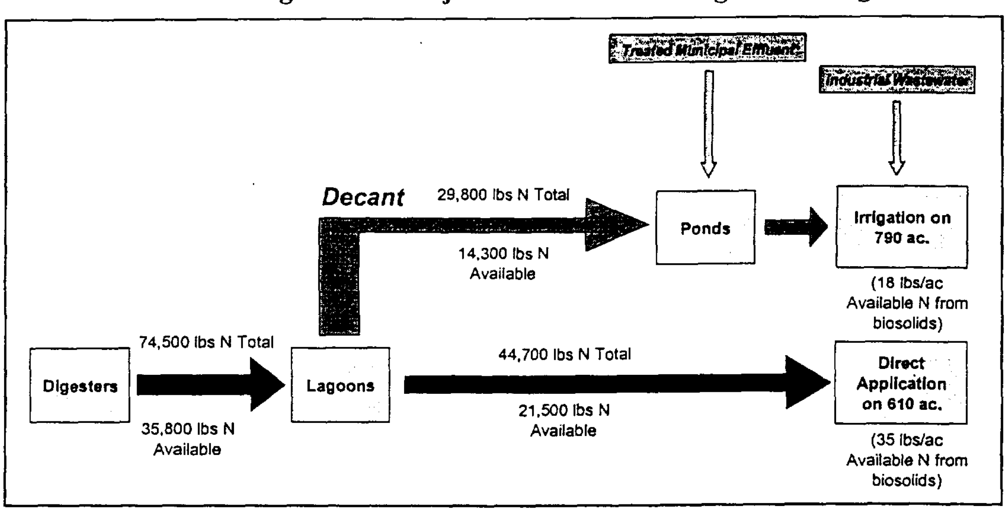
Year	Volume,[a] Mgal	Solids,[b] %	NH_3, lbs	Available NH_3, lbs	Organic N, lbs	Available Organic N,[c] lbs	Total N Available,[d] lbs
1997	5.8	3.0	42,180	21,100	18,340	8,250	29,600
1998	5.6	2.9	24,700	12,400	20,260	9,120	21,500
1999	6.0	3.0	24,700	12,400	39,600	17,800	30,700
Average	5.8	2.9	30,500	15,300	26,100	11,700	27,300
Projected	7.6	3.0	40,000	20,000	34,200	15,400	35,800

(a) Measured by change in biosolids lagoon water level during the pumping of biosolids to fields.
(b) Measured from grab samples of settled biosolids in the biosolids lagoon.
(c) Available N during the first 10 years after biosolids application.
(d) Total nitrogen available for plant uptake, includes small amounts of nitrate in biosolids lagoon influent.

be negligible. The total amount of available organic nitrogen was estimated at 45 percent of applied organic nitrogen. The variability in loadings from year to year may have been partly due to the limited number of grab samples taken. The average loadings were used as the basis for future projections.

Treatment plant staff estimate that about 40 percent of the liquid volume of digested biosolids sent to the biosolids lagoon is decanted to the irrigation storage ponds where it is blended with municipal and industrial effluent for irrigation. The remaining 60 percent is directly pumped and land applied through the surface irrigation system. Approximately 350 acres of the total 790 acres available are used for biosolids application each year. The fields receiving biosolids directly are rotated every year so that Fields 1 through 5 (610 acres, see Figure 8-2) receive about the same average loadings over time. There are normally three to seven applications of biosolids per year between late Spring and mid-Fall. Fields 6A through 6G east of Interstate 5 have not received biosolids. Figure 9-1 below schematically shows the fate of biosolids and the associated nitrogen assuming current available fields for irrigation and biosolids application.

Figure 9-1. Projected Biosolids Nitrogen Loadings



Comparison with Acceptable Loading Rates

The typical nitrogen utilization rate for alfalfa is 480 pounds/acre and the rate for a corn/oats double crop rotation is 365 pounds/acre. The nitrogen loading rate and available nitrogen is calculated every year for every field. The annual reporting forms for 1998 and 1999 are contained in Appendix [redacted]. The projected available nitrogen loading of 53 pounds/acre (Figure 9-1) is substantially less than the potential nitrogen uptake of the crops. Additional discussion of nitrogen loading rates is presented in Section 10.

The nitrogen loading rate calculations assume uniform application of nitrogen from the biosolids. Based on visual observations only, wastewater treatment plant staff estimate that roughly 25 percent of the solids are deposited in the first 10 percent of the furrows. This percentage is somewhat higher for applications to cut alfalfa. While the uniformity of application of solids and organic nitrogen may be marginal, the uniformity of application of dissolved ammonia is probably reasonably good. Since the projected available nitrogen loading rate for the organic nitrogen associated with the solids is only 30 pounds/acre, or 10 percent of average annual crop uptake, the mediocre solids distribution uniformity does not appear to be a large factor for overloading nitrogen at the upper ends of fields.

BIOSOLIDS DISPOSAL/REUSE ALTERNATIVES

Biosolids disposal/reuse alternatives include continuing with the current system, modifying the existing system, tanker truck injection of liquid biosolids, dewatering and spreading dewatered biosolids, and composting or co-composting.

Existing System

This system has worked effectively for the City of Lodi, but with increased quantities the uneven solids distribution may need to be investigated in greater detail to satisfy regulatory agencies.

Modified Surface Irrigation

There are three alternatives to the existing method of applying biosolids.

1. One alternative would be to agitate and mix the solids with a greater amount of treated effluent. This would suspend the solids more thoroughly and provide greater application velocities to reduce the settling at the upper end of the field.

2. The second alternative would be to provide level basins for irrigation and apply the biosolids with the normal irrigations. The level basin irrigation alternative requires high flow rates and greater velocity than graded border or furrow irrigation, thus pushing the solids further out into the field and improving distribution of both solids and nitrates.

3. The third alternative would be to apply the biosolids in surge flow surface irrigation systems. This would provide greater instantaneous velocities for better application distribution.

For alternatives which involve furrow irrigation, straight furrows would be preferred over furrows which have multiple turns.

Tanker Injection

Acquisition of a tanker truck for application of biosolids to the land would provide for the optimum in consistency of application rate for both solids and nitrogen compounds. The truck could be provided for liquid application or injection with no further treatment required for the biosolids from the lagoon. This alternative would provide uniform distribution of biosolids and would work effectively on alfalfa fields with the use of "grass shanks." These shanks inject the liquid biosolids with little damage to the alfalfa crop. A 3,100-gallon unit can be acquired for about $200,000. This unit has a diesel engine that consumes fuel at a rate of 6 gallons/hour. The unit is equipped with high flotation tires that would minimize compaction of the soils. Additional associated facilities for loading, vehicle storage, *etc.*, would add about $100,000 in capital cost. Approximately 1,500 trips per year would be required to dispose of the projected biosolids after decanting. At 30 minutes per trip, this would translate to about 0.5 additional staff for driving and about 0.2 staff for support. Total O&M costs are estimated at approximately $60,000 per year. This work could also be contracted to eliminate the capital cost while increasing the O&M costs.

Dewatered Biosolids Application

A conventional agricultural spreader truck could be purchased for application of dewatered solids. Applications would be limited to times when the fields are being tilled (spring and fall for the corn/oats rotation or fall for new alfalfa fields). A major disadvantage of this alternative is the added cost of dewatering the biosolids, which would be very substantial. For a single belt filter press operating 500 hours per year, the capital costs including support facilities would be on the order of $1.5 to $2.0 million based on costs at San Bernardino, Fresno, and Sacramento Regional wastewater plants. The total O&M costs would be approximately $40 to $50 per dry ton, or $250,000 per year. Sludge drying lagoons could be a lower cost dewatering alternative as is discussed later in Section 11, but the costs of dewatering would still be a substantial increase over current practices. Costs for actual spreading of dewatered solids would be somewhat less than for the tanker injection alternative, or about $40,000 per year.

Compost Biosolids to Provide Class A Material

For biosolids to receive a Class A designation, they must be treated by an approved process to significantly reduce pathogens such as composting or thermal pasteurization. The composting option provides a product that can be used by homeowners or landscapers for a variety of uses. The possible uses are:

- Sell to wholesalers for use as a soil amendment,
- Bag and sell to homeowners, and/or
- Use as an onsite soil amendment on the sports complex fields and landscaped areas.

The cost of composting varies, depending on the type of system used. The costs range from $20/ton to $50/ton of compost, not including the costs of biosolids dewatering. Dewatering would add another $15 per CY of compost for a 3:1 green waste:biosolids ratio. Composting could utilize a combination of leaves collected by the City, garden waste collected as part of the City's contracted garbage services, and green waste from the City's own landscaped areas. The possible revenue from selling compost depends on the market and how the composted material is sold (bulk or bagged). Revenues vary from $2/CY to $12/CY.

The major difficulty with this alternative is that land application of biosolids is prohibited in the unincorporated areas of San Joaquin County. A major advantage of this alternative is that biosolids can be co-composted with City-collected green waste. Co-composting has been implemented successfully by the City of Modesto as a more cost-effective means to dispose of their city-wide green waste collections than landfill disposal.

THERMAL TREATMENT

The Northern California Power Agency owns and operates a gas-fired power generation facility adjacent to the wastewater treatment facility. This facility has excess heat that could be used to dry biosolids and produce a Class A product that could be used for City parks and recreation facilities. Grant money may be available for both parties to design and operate a pilot facility that would produce a pelletized material that could be productively used by the City of Lodi. This pilot study could be completed while continuing with the City's current biosolids operation.

In 1995, West Yost & Associates completed an evaluation of drying and disinfecting biosolids using heat from cogeneration[2]. The analysis was for biosolids generated from the treatment of 7 Mgd of wastewater at the City of Vacaville's Easterly Wastewater Treatment Plant. The facilities required for the drying process included belt filter presses, an Envirex brand heated sludge dryer, a building, and associated facilities. The capital and O&M costs without the belt filter press were $1.6 million and $102,000 per year, respectively, in 1994 dollars. Translated to Year 2005 dollars for 8.5 Mgd, the costs would be $2.0 million capital and $130,000 annual O&M. Including belt filter press pre-drying would nearly double these costs. Unless grant funds became available, the thermal drying alternative would not be economically competitive with other alternatives.

SUMMARY

The City should continue with its existing procedure of applying biosolids within agronomic use rates in the furrows of the corn crop and on alfalfa fields after the last cutting of alfalfa in the fall. The costs of alternative biosolids distribution methods are not justifiable for the potential improvements in distribution uniformity based on an evaluation of nitrogen loading rates. However, some relatively low cost improvements should be made to the existing operations to improve uniformity.

Application scheduling should be adjusted to match times when the fields have been harvested and are about to be tilled for the subsequent crop. Biosolids should be agitated and mixed with additional effluent to suspend the solids more thoroughly. Irrigation should be performed with straight furrows and higher flow rates per furrow or basin to "push" the solids further down the field. The result would be more even distribution of solids and nitrogen compounds.

REFERENCES

[1] U.S. EPA. *Process Design Manual for Utilization of Municipal Wastewater Sludge on Land.* EPA 625/1-83-016, October 1983.

[2] West Yost & Associates. *Digester Biosolids Disinfection Study.* Prepared for the City of Vacaville, June 6, 1995.

SECTION 10. ALTERNATIVES FOR EFFLUENT LAND TREATMENT, REUSE, AND STORAGE

INTRODUCTION

The purpose of this section is to present the further development and evaluation of alternatives for the disposal, reuse and/or storage of treated municipal effluent, industrial wastewater, and biosolids. The first evaluation presented in this section is for hydraulic loadings management, *i.e.* where to send all the effluent. Sizing criteria for the land treatment, reuse, and storage facilities are developed from the results of the hydraulic loading management evaluation. Other issues related to loading rates such as nitrogen loadings and groundwater mounding are then also evaluated to determine sizing criteria. The limiting sizing criteria are then used to develop initial cost estimates for the facilities. Combined treatment and disposal/reuse alternatives are evaluated in Section 13 EVALUATION OF TREATMENT AND DISPOSAL/REUSE PROCESS TRAIN ALTERNATIVES. Other issues related to industrial wastewater treatment and disposal/reuse, and biosolids disposal/ reuse were discussed previously in Sections 8 and 9.

OPERATIONAL SCENARIOS

There are three basic operational scenarios based on the anticipated discharge requirements and the combinations of alternatives developed in Section 5:

1. Complete surface discharge (Section 5, Alternative Combinations DC-D, DC-W, BC-D, BC-W)
2. Partial discharge to Bishop Cut (Section 5, Alternative Combination BC-PD)
3. Land discharge (Section 5, Alternative Combination LD)

Operational Scenarios 2 and 3 involve issues and constraints related to farming operations and the potential Sports Complex.

STORAGE RESERVOIRS

To reduce or eliminate discharge to the Delta during portions of the year, it will be necessary to store or percolate dispose of excess effluent. Additional effluent storage reservoirs could be constructed on City-owned land to provide this additional storage capacity.

Existing Storage

When not discharging to Dredger Cut, treated effluent is conveyed to the 9 acre-feet (af) equalization pond by gravity flow and then either pumped to the irrigation system or into the effluent storage ponds until needed for irrigation.

There are four existing storage ponds located north of the treatment plant as shown on Figure 10-1. The combined area of these ponds is 41 acres. The combined volume is 370 af. The maximum water depth in the ponds is approximately 9 feet with a 1-foot freeboard.

Future Storage Reservoir Locations

New storage reservoirs could be located west of the existing ponds and immediately north or south of the existing 48-inch outfall pipeline to Dredger Cut. Reservoirs at these nearby locations would be easy to hydraulically interconnect with the existing ponds. New ponds adjacent to the existing ponds could share levee sections to save costs.

DEDICATED PERCOLATION BASINS OR FIELDS

Some of the soils north and east of the treatment plant are sandy loam texture, with relatively high percolation rates if deep ripped to break up the underlying hard pan. Relatively high amounts of effluent could be disposed of through percolation if properly managed. At a minimum, this would require that fields be fallow and have very flat slopes with intermediate berms or checks to maintain continued ponding for several days at a time. Surface drainage and runoff return facilities would also be required. Based on infiltration tests, soil borings, and proximity to the existing facilities, the most desirable areas for percolation basins are shown in Figure 10-1. City property and nearby land south of the areas shown on Figure 10-1 would not be suitable for winter percolation disposal of effluent.

Classic rapid infiltration basins could be constructed at dead level slopes with containment berms to provide the greatest percolation disposal capacity, but at a relatively high initial cost. Dedicated percolation fields or basins could accept some industrial wastewater and biosolids, although the solids would reduce percolation rates and require more frequent disking. The major disadvantage of percolation fields or basins is that those areas would not be available for farming during much or all of the year, resulting in a loss of farming revenue.

The typical design criteria is based on 2 to 4 percent of infiltration test results[1], which translates to 0.8 to 1.7 inches per day for the 35 min/in average double ring test infiltration rate in the area shown in Figure 10-1 (see soils report, Appendix [redacted]). The planning level percolation rate assumed for the percolation fields or basins was 1 inch per day. This is within the acceptable design range based on soils information and is relatively low compared to the loading rate for most classical rapid infiltration systems.

The operation of percolation basins can have an effect on the amount of nitrogen in percolate, groundwater mounding, mosquito control, maintenance, and availability for summer crops. The EPA has published recommendations for optimizing nitrogen removal in rapid infiltration basins[2,3]. With proper wetting and drying cycle durations and sufficient carbon, it should be possible to remove about 50 percent of the applied nitrogen. Excessively long ponding durations can create undesirable mosquito breeding conditions. Groundwater level considerations can dictate maximum allowable ponding durations and the availability of basins or fields for summer cropping. Hydraulic and nitrogen loading rates are discussed in greater detail later in this section.

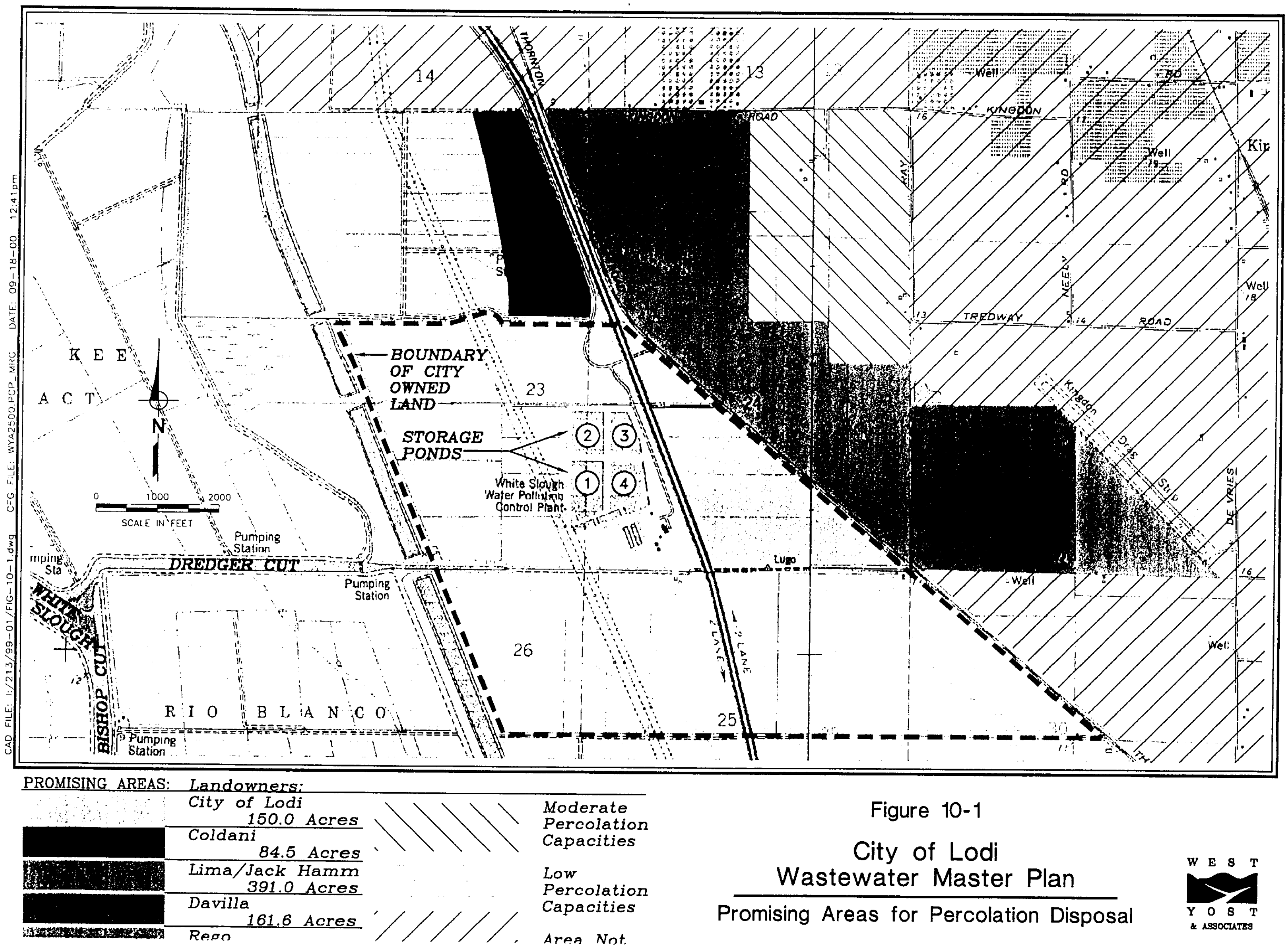


Figure 10-1

City of Lodi
Wastewater Master Plan

Promising Areas for Percolation Disposal

CROP IRRIGATION

A total of 790 acres of land is currently capable of being irrigated with municipal effluent, industrial wastewater, and biosolids out of a total of 880 acres of irrigated farm land owned by the City. The 90 acres furthest east is not currently irrigated with effluent, but could be if new conveyance facilities were constructed. The proposed Sports Complex would reduce the amount of farmland to 480 acres and add approximately 250 acres of turf grass.

Alfalfa

Alfalfa is already grown on much of the City's existing land. It is a multi-year crop with a high water use rate, high nitrogen uptake rate, and value to the nearby dairies. As discussed in Sections 8 and 9, alfalfa is not an ideal crop for accepting biosolids or industrial wastewater, especially using border strip surface irrigation. With border strip irrigation, solids tend to be concentrated at the upper end of the field, resulting in a greater potential for odors, crop impacts, and groundwater impacts.

Turf Grass

If the sports complex is constructed, the grass fields will be irrigated with reclaimed water. The water and nitrogen use rates for turf grass can be relatively high. Turf grass area is combined with alfalfa in water balance calculations because they have essentially the same crop evapotranspiration coefficient (1.0). Turf grass is not suitable for irrigation application of high solids content wastewater such as the industrial wastewater or biosolids.

Annual Crops

Many annual crops are grown in the area surrounding the wastewater treatment plant. The main annual crops grown on the land owned by the City are corn and oats. Corn has a high water use rate in mid-summer, but a decreasing water use rate as harvest time approaches in the fall. Oats are planted as a winter crop, so they only need irrigation after planting in the fall and possibly during a dry spring. The annual crops are important for the application of high solids content wastewater. With furrow irrigation, industrial wastewater or biosolids can be applied at a reasonable uniformity. Industrial wastewater and biosolids can also be readily land applied to fallow ground in the fall after the harvest of annual crops.

CONSTRUCTED WETLANDS

Constructed free water surface wetlands were discussed previously in Section 7. Constructed wetlands would use slightly more water than grass crops per unit area and would also provide some temporary storage capacity.

OTHER USES OF RECLAIMED WATER

Mosquito Fish Ponds

The San Joaquin County Mosquito and Vector Control District maintains ponds just south of the treatment plant for raising mosquito fish (gambuzia). The ponds are supplied with effluent from the treatment plant. The monthly range of water usage during the last two years has been 0 to 333,000 gpd, with an average usage of 97,000 gpd. Due to their operations, the water usage does not appear to follow a seasonal pattern.

NCPA Power Plant Water

The Northern California Power Agency runs a natural gas-fired 49-megawatt power plant west of the biosolids lagoon. The power plant is primarily run to satisfy peak electrical demands. The power plant receives treatment plant effluent, filters it, and uses it for cooling water. Water used for steam generation receives the same treatment plus reverse osmosis. The power plant returns most of the blowdown and reject water to the City's treatment plant. The power plant has recently began using a deep injection well to dispose of about 25 percent (100,000 gpd) of the wastewater on an intermittent basis. The monthly range of net water usage during the last two years has been 0 to 388,000 gpd, with an average usage of 167,000 gpd. The water usage is highly variable and does not appear to follow a seasonal pattern. There have been preliminary discussions regarding an expansion of the power plant, which would increase its water usage.

WATER BALANCES

Water balances were prepared using the flow projections from Section 3 for the operational scenarios presented earlier in this Section. The water balances conservatively assume a constant combined usage of 100,000 gpd by the mosquito fish ponds and power plant. The results of the water balances show the minimum required land areas for irrigation, percolation, and wetlands storage. They also show the required storage volumes where storage reservoirs are considered. A 1 in 100-year annual rainfall proportioned by month was applied for all water balances unless indicated otherwise.

Complete Surface Discharge (Alternatives DC-D, DC-W, BC-D, BC-W)

For year-round surface discharge (Alternatives DC-D, DC-W, BC-D, BC-W from Section 5), only industrial wastewater and biosolids would have to be land applied. The biosolids were conservatively assumed to be diluted in an average 0.1 Mgd of effluent prior to application. The water balance for this scenario is presented in Table 10-1. This water balance assumed that biosolids and industrial wastewater are only applied between April and October. The values for annual crops area and percolation field area were derived by iteration based on maintaining complete land disposal/reuse for April through October. Reservoirs and wetlands were not considered for storing industrial wastewater because of potential odor problems. It should be noted that 30 days (approx. 800 af) of reservoir or wetlands storage for treated municipal effluent is needed with this scenario to accommodate times when the dissolved oxygen level in Dredger Cut falls below 5.0 mg/L or when dilution in Bishop Cut is low because of South Delta pumping restrictions. Assuming that most of the existing 300 af of storage could be used for this purpose, an additional 500 af of new storage would be required.

The water balance shows that 350 acres of annual crops and 350 acres of low rate percolation fields are needed for industrial effluent and biosolids disposal. The fallow percolation fields are needed for times when annual crop irrigation demands are low. The water balance does not show any alfalfa because alfalfa cannot be loaded as heavily with cannery effluent as annual crops or

Table 10-1. Water Balance for Land Application of Industrial Wastewater and Biosolids (Alternatives DC-D, DC-W, BC-D, BC-W)

1 in 100 Year Rainfall

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
				Total	Annual Crops		Percolation Fields		
Month	100 Year Rainfall (in./mo)	Reference E.T. (in./mo)	Industrial Inflow (ac-in./mo)	Wastewater Inflow (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Treatment & Discharge (ac-in.)
Jan.	5.79	0.98	173	173	0	0	0	0	173
Feb.	5.44	1.54	143	143	0	0	0	0	143
Mar.	4.96	2.93	256	256	0	0	0	0	256
Apr.	2.72	4.72	161	272	0	0	0	(272)	0
May	1.90	6.22	113	227	664	(1,377)	0	0	0
June	0.46	7.32	272	382	161	(2,639)	0	0	0
July	0.44	8.06	2,181	2,295	155	(4,433)	0	0	0
Aug.	0.41	6.96	4,404	4,518	142	(3,680)	0	(981)	0
Sept.	1.48	5.31	5,158	5,268	518	(2,259)	0	(3,527)	0
Oct.	2.04	3.30	2,498	2,613	714	(1,050)	0	(2,276)	0
Nov.	4.37	1.42	61	61	0	0	0	0	61
Dec.	5.17	0.73	79	79	0	0	0	0	79
Totals (in./yr)	35.2	49.5							

MAIN INPUT PARAMETERS	Annual Crops	Perc. Fields*
Percolation Rates (in./mo.):	3.0	6.0
Evapotranspiration Coefficient:	varies	1.00
Surface Areas (ac.):	350	350
Diluted Biosolids Flow Rate (Mgd):	0.10	

* Note: Indicates low rate percolation fields.

(1) Rainfall data is from Lodi station, using 100 year monthly statistics normalized to 100 year annual total (from Western Reg. Climate Center).
(2) Reference evapotranspiration data is from "Irrigation with Reclaimed Municipal Wastewater - A Guidance Manual," by Pettygrove and Asano, 1985.
(3) Projected industrial wastewater flow rate
(4) (Diluted biosolids and industrial flow rate in Mgd) x 36.83 x (number of days in the month)
(5) Col. 1 x (annual crop area)
(6) - (Col. 2 x (melded monthly evapotranspiration coefficient for annual crops) + (annual crops area percolation rate)) x (annual crops area)
(7) Col. 1 x (percloation field area).
(8) - (Col. 2 x (evapotranspiration coefficient for perc. fields) + (field percolation rate)) x (percolation field area), up to the amount of water left.
(9) Sum of columns 4 through 8

fallow percolation fields (see Appendix [redacted]). The 700-acre net land requirement for hydraulic loading is greater than the 210 acres required for BOD loading for the industrial wastewater specified in Section 8. The 700-acre requirement is less than the 790 acres currently available for irrigation. However, if the Sports Complex were to be constructed, only 480 acres would be available for application of industrial effluent and biosolids. Extra annual crop area above 350 acres would reduce the need for percolation fields by about one-half of the extra annual crop area.

Partial Discharge to Bishop Cut (Alternatives BC-PD, BC-PW)

These alternatives includes the discharge of effluent to Bishop Cut up to the maximum allowable flow rate permissible without tertiary treatment. As was discussed previously in Section 4, it is assumed that a 20:1 dilution in Bishop Cut would be necessary to prevent tertiary treatment requirements from being imposed. 20:1 dilution in Bishop Cut should be available during December through March with normal to wet climatic conditions. During other months of the year or during a dry winter, only a portion of the effluent could be discharged while maintaining a 20:1 dilution. The rest of the effluent would be disposed of in percolation basins and fields, or would be used for agricultural irrigation. For water balance calculations, it was assumed that half the effluent would be sent to Bishop Cut and half to percolation disposal during winter months. This is a reasonable assumption for the net flows in Bishop Cut as estimated in the modeling report contained in Appendix [redacted]. During spring months, the amount of discharge to Bishop Cut would taper off. Essentially all the water would be used for crop irrigation in the summer. The water balance for this scenario is shown in Table 10-2 and Figure 10-2.

Land Discharge (Alternative LD)

For the land discharge scenario, all effluent would be land applied. Based on experience at other sites, providing sufficient reservoir storage for all late fall and winter months would not be a viable alternative because of excessive capital construction cost. Instead, all effluent not used for irrigation would be disposed in dedicated percolation fields or basins. The water balance for this scenario for 1 in 100-year rainfall is shown in Table 10-3. The water balance for average year rainfall is shown in Table 10-4. The distribution of effluent by month is shown graphically in Figure 10-3.

The minimum required areas shown in Table 10-3 were derived by iteration based on having a balance of crop types and no surface discharge. Summer basins refer to winter percolation basins which would have to remain fallow in the summer and fall to provide adequate effluent disposal capacity. The alfalfa area could vary somewhat from the 400 acres shown, but the annual crop area should not be reduced below about 400 acres because annual crop area is better suited to receiving biosolids and industrial wastewater. Some industrial wastewater can be applied to alfalfa if it is blended with municipal effluent and not hydraulically loaded in excess of agronomic rates. The 400 acres shown for percolation fields and basins would be new fields with high permeability soils located generally to the east of the City's current land holdings (Figure 10-1). If percolation basins are selected as an effluent disposal method, pilot testing of percolation disposal should be performed to confirm the assumptions based on initial soils testing (Appendix [redacted].

Figure 10-2. Partial Discharge to Bishop Cut
Effluent Loading by Reuse or Discharge Type
(Average Rainfall, Year 2020)

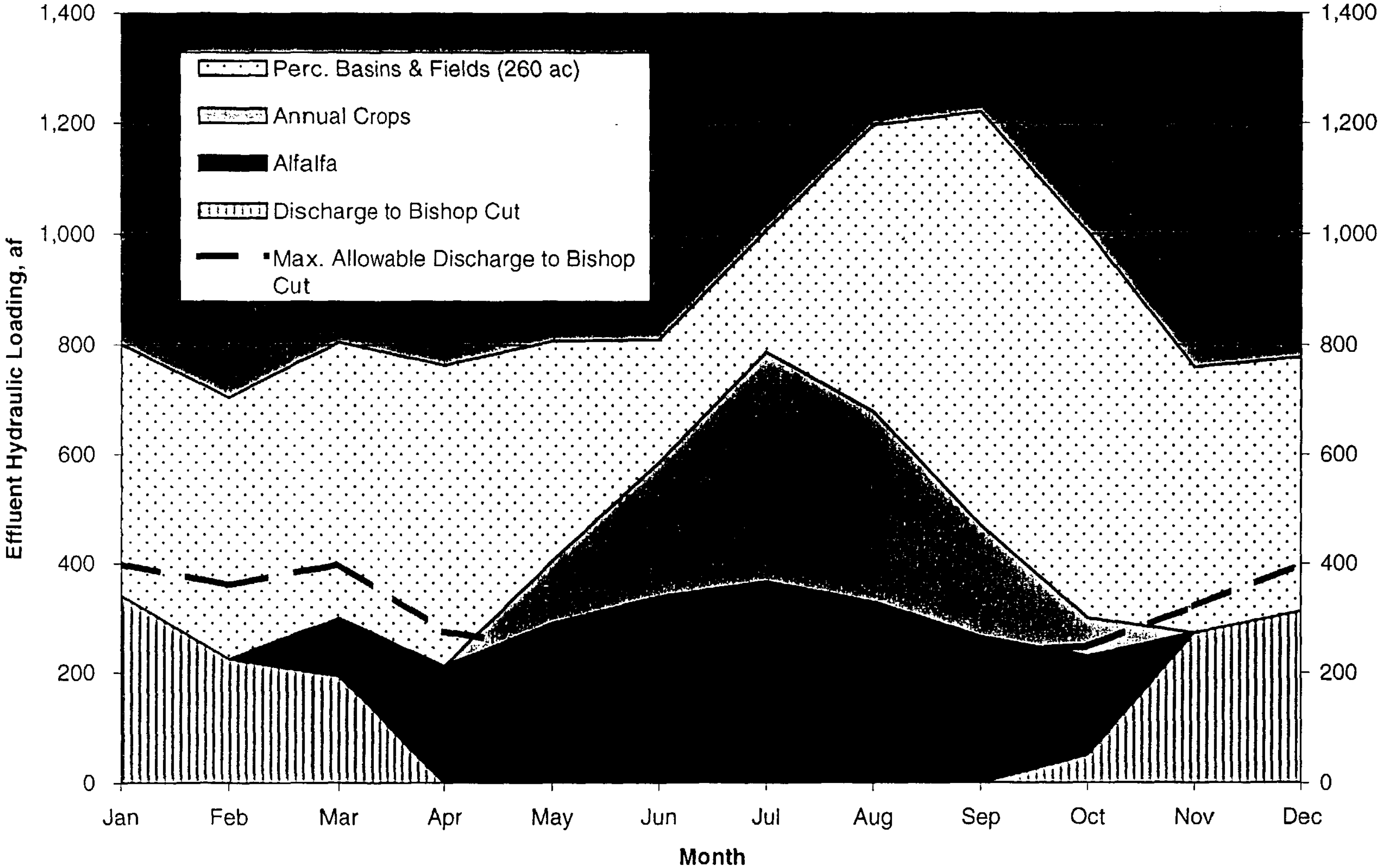


Table 10-2. Water Balance for Partial Discharge to Bishop Cut (Alternatives BC-PD, BC-PW)

Average Rainfall, Year 2020

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
				Total	Alfalfa & Grass		Annual Crops			Percolation Fields & Basins			Max.
Month	Average Rainfall (in./mo)	Reference E.T. (in./mo)	Industrial Inflow (ac-in./mo)	Wastewater Inflow (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Water Left (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Discharge (ac-in.)	Allowable Discharge (ac-in.)
Jan.	3.52	0.98	173	9,601	0	0	0	0	9,601	704	(6,195)	4,110	4,795
Feb.	2.85	1.54	143	8,445	0	0	0	0	8,445	570	(6,309)	2,707	4,331
Mar.	2.73	2.93	256	9,647	1,092	(2,372)	0	0	8,367	546	(6,586)	2,327	4,795
Apr.	1.34	4.72	161	9,154	536	(3,090)	0	0	6,600	348	(6,949)	0	3,315
May	0.50	6.22	113	9,680	200	(3,690)	200	(1,573)	4,816	130	(4,946)	0	2,968
June	0.14	7.32	272	9,702	56	(4,129)	56	(3,016)	2,668	36	(2,705)	0	2,873
July	0.05	8.06	2,181	12,056	20	(4,422)	20	(5,066)	2,607	13	(2,620)	0	2,968
Aug.	0.05	6.96	4,404	14,343	20	(3,983)	20	(4,205)	6,195	13	(6,208)	0	2,968
Sept.	0.32	5.31	5,158	14,644	128	(3,326)	128	(2,582)	8,992	83	(9,076)	0	2,873
Oct.	0.90	3.30	2,498	12,029	360	(2,518)	360	(1,200)	9,031	234	(8,657)	609	2,968
Nov.	2.29	1.42	61	9,106	0	0	0	0	9,106	458	(6,283)	3,280	3,867
Dec.	2.92	0.73	79	9,322	0	0	0	0	9,322	584	(6,146)	3,760	4,795
Totals (in./yr)	17.6	49.5											

Average Hydraulic Loading (in/mo) 25.69

MAIN INPUT PARAMETERS

		Alfalfa	Annual Crops	Winter Basins	Summer Basins
Max. Percolation Rates (in./mo.):		3.0	3.0	30.0	30.0
Evapotranspiration Coefficient:		1.0	varies	1.0	1.0
Surface Areas (ac.):		400	400	200	260
Municipal Effluent Flow (Mgd):	8.50				
Mosquitofish and Power Plant Usage (Mgd):	0.10				

(1) Rainfall data is from Lodi station.
(2) Reference evapotranspiration data is from "Irrigation with Reclaimed Municipal Wastewater - A Guidance Manual," by Pettygrove and Asano, 1985.
(3) Projected industrial wastewater flow rate
(4) (Wastewater flow rate in Mgd) x 36.83 x (number of days in the month)
(5) Col. 1 x (alfalfa+grass area)
(6) - (Col. 2 x (evapotranspiration coefficient for alfalfa) + (alfalfa area deep percolation rate)) x (alfalfa area)
(7) Col. 1 x (annual crop area)
(8) - (Col. 2 x (melded monthly evapotranspiration coefficient for annual crops) + (annual crops area percolation rate)) x (annual crops area)
(9) Sum of Columns 4 through 8
(10) Col. 1 x (percloation basins area).
(11) - (Col. 2 x (evapotranspiration coefficient for perc. basins) + (basins percolation rate)) x (perc. basin area).
(12) Sum of columns 9 through 11
(13) Maximum allowable discharge to maintain 20:1 dilution for more than 85% of the years in the Bishop Cut net flow modeling

Table 10-3. Water Balance for Land Discharge (Alternative LD)

1 in 100 Year Rainfall, Year 2020

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
				Total	Alfalfa & Grass		Annual Crops			Percolation Fields & Basins		
Month	100 Year Rainfall (in./mo)	Reference E.T. (in./mo)	Industrial Inflow (ac-in./mo)	Wastewater Inflow (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Water Left (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Discharge (ac-in.)
Jan.	5.79	0.98	173	9,601	0	0	0	0	9,601	2,314	(11,916)	0
Feb.	5.44	1.54	143	8,445	0	0	0	0	8,445	2,175	(10,621)	0
Mar.	4.96	2.93	256	9,647	1,982	(2,372)	0	0	9,257	1,982	(11,239)	0
Apr.	2.72	4.72	161	9,154	1,089	(3,090)	0	0	7,153	871	(8,024)	0
May	1.90	6.22	113	9,680	759	(3,690)	759	(1,573)	5,934	607	(6,541)	0
June	0.46	7.32	272	9,702	184	(4,129)	184	(3,016)	2,924	147	(3,071)	0
July	0.44	8.06	2,181	12,056	177	(4,422)	177	(5,066)	2,921	141	(3,062)	0
Aug.	0.41	6.96	4,404	14,343	163	(3,983)	163	(4,205)	6,480	130	(6,610)	0
Sept.	1.48	5.31	5,158	14,644	592	(3,326)	592	(2,582)	9,919	473	(10,393)	0
Oct.	2.04	3.30	2,498	12,029	815	(2,518)	815	(1,200)	9,942	652	(10,595)	0
Nov.	4.37	1.42	61	9,106	0	0	0	0	9,106	1,749	(10,854)	0
Dec.	5.17	0.73	79	9,322	0	0	0	0	9,322	2,069	(11,391)	0
Totals (in./yr)	35.2	49.5										

Average Hydraulic Loading (in/mo) 24.83

MAIN INPUT PARAMETERS

		Alfalfa	Annual	Winter Basins	Summer Basins
Max. Percolation Rates (in./mo.):		3.0	3.0	30.0	30.0
Evapotranspiration Coefficient:		1.0	varies	1.0	1.0
Surface Areas (ac.):		400	400	400	320
Municipal Effluent Flow (Mgd):	8.50				
Mosquitofish and Power Plant Usage (Mgd):	0.10				

(1) Rainfall data is from Lodi station, using 100 year monthly statistics normalized to 100 year annual total (from Western Reg. Climate Center).
(2) Reference evapotranspiration data is from "Irrigation with Reclaimed Municipal Wastewater - A Guidance Manual," by Pettygrove and Asano, 1985.
(3) Projected industrial wastewater flow rate
(4) (Wastewater flow rate in Mgd) x 36.83 x (number of days in the month)
(5) Col. 1 x (alfalfa+grass area)
(6) - (Col. 2 x (evapotranspiration coefficient for alfalfa) + (alfalfa area deep percolation rate)) x (alfalfa area)
(7) Col. 1 x (annual crop area)
(8) - (Col. 2 x (melded monthly evapotranspiration coefficient for annual crops) + (annual crops area percolation rate)) x (annual crops area)
(9) Sum of Columns 4 through 8
(10) Col. 1 x (percloation basins area).
(11) - (Col. 2 x (evapotranspiration coefficient for perc. basins) + (basins percolation rate)) x (perc. basin area).
(12) Sum of columns 9 through 11

Table 10-4. Water Balance for Land Discharge - Avg. Year (Alternative LD)

Average Rainfall, Year 2020

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
				Total	Alfalfa & Grass		Annual Crops			Percolation Fields & Basins		
Month	Average Rainfall (in./mo)	Reference E.T. (in./mo)	Industrial Inflow (ac-in./mo)	Wastewater Inflow (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Water Left (ac-in./mo)	Rainfall (ac-in./mo)	Evap. & Perc. (ac-in./mo)	Discharge (ac-in.)
Jan.	3.52	0.98	173	9,601	0	0	0	0	9,601	1,408	(11,009)	0
Feb.	2.85	1.54	143	8,445	0	0	0	0	8,445	1,140	(9,585)	0
Mar.	2.73	2.93	256	9,647	1,092	(2,372)	0	0	8,367	1,092	(9,459)	0
Apr.	1.34	4.72	161	9,154	536	(3,090)	0	0	6,600	429	(7,029)	0
May	0.50	6.22	113	9,680	200	(3,690)	200	(1,573)	4,816	160	(4,976)	0
June	0.14	7.32	272	9,702	56	(4,129)	56	(3,016)	2,668	45	(2,713)	0
July	0.05	8.06	2,181	12,056	20	(4,422)	20	(5,066)	2,607	16	(2,623)	0
Aug.	0.05	6.96	4,404	14,343	20	(3,983)	20	(4,205)	6,195	16	(6,211)	0
Sept.	0.32	5.31	5,158	14,644	128	(3,326)	128	(2,582)	8,992	102	(9,095)	0
Oct.	0.90	3.30	2,498	12,029	360	(2,518)	360	(1,200)	9,031	288	(9,319)	0
Nov.	2.29	1.42	61	9,106	0	0	0	0	9,106	916	(10,022)	0
Dec.	2.92	0.73	79	9,322	0	0	0	0	9,322	1,168	(10,490)	0
Totals (in./yr)	17.6	49.5										

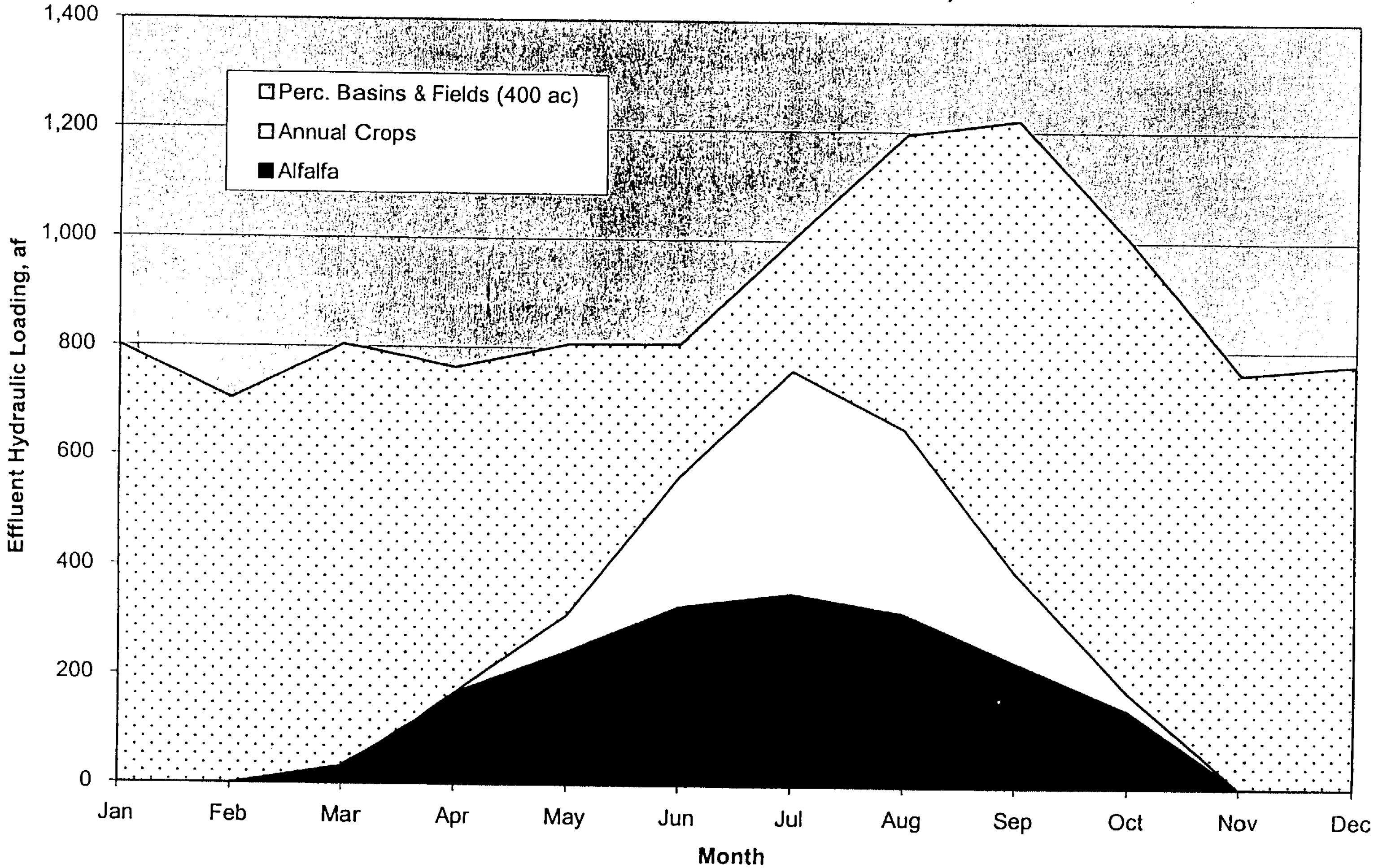
Average Hydraulic Loading (in/mo) 21.96

MAIN INPUT PARAMETERS

		Alfalfa	Annual Crops	Winter Basins	Summer Basins
Max. Percolation Rates (in./mo.):		3.0	3.0	30.0	30.0
Evapotranspiration Coefficient:		1.0	varies	1.0	1.0
Surface Areas (ac.):		400	400	400	320
Municipal Effluent Flow (Mgd):	8.50				
Mosquitofish and Power Plant Usage (Mgd):	0.10				

(1) Rainfall data is from Lodi station.
(2) Reference evapotranspiration data is from "Irrigation with Reclaimed Municipal Wastewater - A Guidance Manual," by Pettygrove and Asano, 1985.
(3) Projected industrial wastewater flow rate
(4) (Wastewater flow rate in Mgd) x 36.83 x (number of days in the month)
(5) Col. 1 x (alfalfa+grass area)
(6) - (Col. 2 x (evapotranspiration coefficient for alfalfa) + (alfalfa area deep percolation rate)) x (alfalfa area)
(7) Col. 1 x (annual crop area)
(8) - (Col. 2 x (melded monthly evapotranspiration coefficient for annual crops) + (annual crops area percolation rate)) x (annual crops area)
(9) Sum of Columns 4 through 8
(10) Col. 1 x (percloation basins area).
(11) - (Col. 2 x (evapotranspiration coefficient for perc. basins) + (basins percolation rate)) x (perc. basin area).
(12) Sum of columns 9 through 11

Figure 10-3. Land Discharge Alternative
Effluent Loading by Field Type
(1 in 100 Year Rainfall, Year 2020)



Summary of Areas and Volumes Required for Water Balances

The areas and volumes required for the various alternatives to satisfy water balances and provide emergency storage or percolation disposal capacity are summarized in Table 10-5. The land areas shown in the water balances and Table 10-5 are net areas, not including roads, berms, or buildings.

Table 10-5. Area and Volume Requirements

Alt.	Alfalfa and Grass, ac	Annual Crops, ac	Winter Percolation, ac	Summer/Fall Percolation, ac	New Storage Reservoirs, ac	New Storage Volume, af	Wetlands, ac	Total, ac
DC-D	0	350	0	350	80	500	—	780
DC-W	0	350	0	350	40	250	130	870
BC-D	0	350	0	350	0	0	0	700
BC-W	0	350	0	350	0	0	130	830
BC-PD	400	400	200	260	0	0	0[b]	1,060
LD	400	400	400	320[a]	0	0	0[b]	1,200

[a] High rate dedicated fields or basins (up to 30 in/mo loading).
[b] Could use wetlands for denitrification in lieu of conventional denitrification facilities.

ALTERNATIVE CROPS

Wood Tree Crops

Poplar and eucalyptus trees are farmed commercially in the Central Valley of California for use in paper and wood products. These tree crops have a relatively high water and nutrient usage rate over a long season. Another advantage of these tree crops is that they are multi-year and require less management than annual crops or alfalfa. Tree crops can be furrow irrigated, allowing some biosolids and industrial wastewater to be applied. Tree crops would require about 10 percent less area than the combined alfalfa and annual crop areas shown in Table 10-5. Some tree crops may be sensitive to oxygen deprivation with high loadings of industrial wastewater.

Greenchopped Corn

The corn currently grown on the site is allowed to dry in the field and is then harvested as grain. There is little or no irrigation of the corn during this drying period. Alternatively, the corn could be harvested green in late August or early September and used for silage. This would allow effluent and biosolids to be applied to the harvested fields during the critical months of September and October. The net effect of this would be to increase the allowable September and October hydraulic loadings by about 5 inches per month for greenchopped corn fields which had already been harvested before September. This could reduce the amount of percolation field area by roughly half the amount of greenchopped corn area. For example, if 200 acres of greenchopped corn were planted, the amount of percolation fields shown in Table 10-1 could be reduced by 100 acres. This could also reduce the amount of summer percolation basin area by about one sixth the amount of greenchopped corn area.

Crops for Human Consumption

If tertiary treatment facilities are constructed, essentially any crop could be irrigated with the treated effluent. Although other crops would not use any more water than alfalfa or tree crops, the City could potentially receive more revenue from its land and water because of the ability to grow higher value crops. Some of the higher value crops grown in the area include fresh market tomatoes, grapes, and vegetables. Tertiary treated effluent could also be used without restriction on land not owned by the City. This could provide additional flexibility in using the treated effluent.

NITROGEN LOADING TO GROUNDWATER

The application of sources of nitrogen to land can be a concern because of the potential for increasing the concentration of nitrate in groundwater. The primary maximum contaminant limit for nitrate-nitrogen is 10 mg/L as N. The background nitrate-nitrogen concentration has recently ranged from 5 to 40 mg/L in the monitoring wells. The concentration of nitrate in percolate would need to be kept below the drinking water limit of 10 mg/L (as N) and may even need to be several mg/L below 10 to satisfy possible future directives by the Regional Water Quality Control Board.

Total nitrogen in treated secondary effluent is primarily nitrate (7.7 mg/L avg), with small amounts of ammonia (1.2 mg/L avg) and nitrite (avg 0.5 mg/L avg). At the 8.5 Mgd design flow rate, the total nitrogen loading rate would be slightly less than 700 lbs/d.

Nitrate is a mobile ion in soil which can leach down to groundwater if it is not taken up by crop roots or denitrified. Nitrogen uptake varies by crop, but is generally in the range of 100 to 480 pounds per acre per year. Denitrification in soil is a function of the amount of carbonaceous matter available in the soil and the development of anoxic conditions.

Biosolids typically contain ammonia and a high amount of organic nitrogen. A portion of the organic nitrogen is mineralized to ammonia over time, which in turn can be oxidized to nitrate. The total nitrogen loading in biosolids is projected to be 49,000 pounds per year, of which approximately 42 percent, or 21,000 net pounds per year, is estimated to be converted into plant available ammonia and nitrate in the soil (see Section 9).

The industrial wastewater is primarily from fruit and tomato canning. This type of wastewater typically contains a very high amount of organic carbon in relation to nitrogen. In intermittently flooded conditions, almost all of the nitrogen will typically become immobilized or denitrified[4] for carbon:nitrogen ratios over 20:1. Therefore the industrial wastewater poses little or no threat to groundwater nitrate concentrations. If blended with secondary effluent and/or biosolids, the industrial wastewater should actually reduce the total amount of available nitrate-nitrogen.

The considerations for minimizing nitrate impacts to groundwater for the land discharge scenarios are discussed below.

Complete Surface Discharge

For surface discharge of almost all treated effluent to either Dredger Cut or to Bishop Cut, the only sources of nitrogen to be applied to land would be from industrial wastewater and/or biosolids. Assuming 350 acres of annual crops, the average annual crop uptake of nitrogen would be approximately 100,000 lbs/year assuming a corn/oats double cropping. This is greatly in excess of the projected plant available nitrogen loading of 21,000 lbs/yr from the biosolids. With the denitrifying effect of the high carbon content of the industrial wastewater, the actual effective loading rate would probably be much less. Therefore there would be a nitrogen deficit for the annual crops, and supplemental fertilizer would need to be applied to achieve satisfactory crop yields. Even if some municipal effluent continues to be used for agricultural irrigation, the total nitrogen applied would be less than the crop uptake.

Irrigation of 400 acres of alfalfa or grass with effluent during the months of March through October would result in a net plant available nitrogen loading rate of 115 pounds per acre per year. Irrigation of 400 acres of annual row crops with effluent and biosolids during May through October would result in a net plant available nitrogen loading rate of approximately 130 lbs/ac/yr. The loading rate to the fallow fields would be approximately 300 pounds per acre per year, with most occurring during April, May, September, and October. For both annual crops and for alfalfa, the projected loading rate to cropped fields is significantly less than the annual crop usage.

The effluent directed to percolation basins should have a low enough nitrogen content or enough carbon such that the concentration of nitrate in deep percolate is below the 6 mg/L discussed previously. The methods for maintaining a low concentration of nitrate in percolate from percolation basins or percolation fields are discussed below under the land discharge alternative.

Partial Discharge to Bishop Cut and Land Discharge

For Alternatives BC-PD, BC-PW, and LD, effluent would have to be applied to percolation basins or fields. For the purposes of this Master Plan, the average concentration of nitrate in percolate is assumed to be 6 mg/L or less.

There are four options for minimizing the nitrate concentration in percolate:

1. Conventional Denitrification
2. Denitrification in Wetlands
3. Blending with Industrial Wastewater
4. Operation of Percolation Fields to Maximize Denitrification

With the historical average concentration of total nitrogen in the effluent of 9.3 mg/L, about one third of the nitrate in the effluent will need to be removed using one or more of the 4 options above.

Option 4 alone would remove approximately 20 to 50 percent of total nitrogen. Nitrogen removal rates would tend to be higher in the summer months and lower in the winter months. Option 3 in combination with Option 4 would increase nitrogen removal substantially in late fall and winter months, but the solids from the industrial wastewater could reduce the infiltration capacity of the percolation basins. Option 2 would remove over 80 percent of the nitrate-nitrogen assuming

130 acres of wetlands. Even 65 acres of wetlands would remove about 50 percent of nitrate-nitrogen. Conventional denitrification (Option 1) can remove up to about 50 percent of nitrate without a supplemental carbon source.

Option 4 should be marginally adequate. Depending upon actual field performance and future discharge requirements, a nitrate reduction of approximately 2 to 3 mg/L using either Option 1 or 2 may be necessary in addition to implementing Option 4. The costs and other considerations of Options 1 and 2 will be compared later in Section 13. Option 3 is appealing for its simplicity, but would need to be pilot tested to determine its viability in percolation basins or dedicated percolation fields.

The effluent and biosolids applied to cropped fields would have essentially the same loading rates and nitrogen usage as with current effluent irrigation practices. Therefore there would be no adverse impacts to groundwater nitrate concentrations from effluent irrigation of crops for the Bishop Cut partial discharge or the land discharge alternative.

GROUNDWATER LEVEL IMPACTS FOR PERCOLATION BASINS

Depending upon the layout of percolation basins or fields, groundwater mounding may need to be controlled for the Bishop Cut partial discharge and the land discharge scenarios. The most likely area for percolation basins or fields for the land discharge scenario was shown in Figure 10-1. The characteristics of the soils are described in the soils report in Appendix [redacted].

The depth to groundwater in the spring for this area is about 10 to 15 feet below ground surface (bgs). Some of the borings show lenses of lower permeability soils, especially at around 10 feet bgs. The few available deeper well and boring logs show sandier strata below 10 feet.

Groundwater mounding could develop above the lower permeability lenses due to the effluent percolation. The groundwater mound elevation could reach an equilibrium if the additional hydraulic head of the mound is enough to force the water through the lower permeability layers. Deep ripping of the basins could also be performed to fracture the lower permeability layers and increase vertical hydraulic conductivity. Any mounding would be less pronounced if the basins are laid out in a long, narrow fashion. Pilot testing and deeper borings will be necessary to confirm the magnitude of groundwater mounding and the effectiveness of these measures.

If groundwater mound equilibrium cannot be achieved with the above measures, underdrains could be required to keep at least a 2-foot unsaturated zone beneath the percolation basins. Theoretical underdrain spacing was evaluated using Hooghoudt's equation, a horizontal hydraulic conductivity of 10 feet/day in the permeable soil, and assuming only half of the applied water percolates through the lower permeability layers. The results indicate that widely spaced (200 to 300 feet) underdrains could control groundwater mounding if needed.

Groundwater collected from underdrains would need to be discharged to the Delta. Although this water would probably be excellent quality except for salinity, some discharge requirements could be imposed. The most similar situation to this is the proposed groundwater level control system at the wastewater irrigation fields for the planned Mountain House community north of Tracy. The adopted permit for discharge of groundwater to Old River does not contain any limitations beyond the standard groundwater quality objectives typically contained in most waste discharge permits (including Lodi's permit).

COST ESTIMATES FOR LAND TREATMENT/REUSE AND DISCHARGE ALTERNATIVES

Preliminary costs for the Land Treatment, Reuse, and Storage Alternatives are shown below in Table 10-6. These costs are based on typical unit costs from other similar facilities. These costs are for comparison purposes only. They include engineering and administrative costs at 20 percent of construction costs and modest contingencies at 40 percent of construction costs. The wetlands costs assume very basic features and only moderate initial planting density. Related improvements in treatment facilities and EIR costs are not included in the table. Costs related to improvements needed for the irrigation and tailwater pumping systems are discussed later in Section 11 because of their proximity to the treatment pond. Combined treatment and disposal/reuse costs are compared in Section 13.

Table 10-6. Preliminary Estimated Costs of Alternatives— Land Treatment/Reuse and Discharge Facilities Only

Alternative	Facility	Capital Costs	Annual O&M	Life Cycle[a]
DC-D	500 af Storage[b]	3,800,000	108,000	4,940,000
DC-W	130 ac Wetlands 250 af Storage[b]	3,900,000[c] 1,900,000	182,000 54,000	
Total, DC-W		5,800,000	236,000	8,300,000
BC-D	7,500 ft Outfall Reaeration, Diffuser[d]	1,600,000 1,100,000	24,000 66,000	
Total, BC-D		2,700,000	90,000	3,650,000
BC-W	100 ac Wetlands Reaeration, Diffuser[d]	3,000,000[c] 1,200,000	182,000 66,000	
Total, BC-W		4,200,000	248,000	6,800,000
BC-PD	260 ac Percolation 7,500 ft Outfall Reaeration, Diffuser[d]	8,320,000[e] 1,600,000 1,100,000	117,000 24,000 66,000	
Total, BC-PD		11,020,000	207,000	13,210,000
BC-PW	260 ac Percolation 60 ac Wetlands Reaeration, Diffuser[d]	8,320,000 1,800,000 1,100,000	117,000 90,000 66,000	
Total, BC-PW		11,220,000	273,000	14,110,000
LD	400 ac Percolation	12,800,000[d]	180,000	14,710,000

[a] Assumes O&M costs converted to present worth costs using 7% interest for 20 years.
[b] In addition to the 300 af of existing storage.
[c] Includes land at $6,000 per acre.
[d] Includes cascade reaeration prior to discharge to Bishop Cut through a submerged outfall diffuser.
[e] Includes land at $8,000 per acre.

SUMMARY AND CONCLUSIONS

The City currently owns sufficient land for disposal/reuse of industrial wastewater, biosolids, and some municipal wastewater. Additional land would be required for alternatives which include wetlands or percolation basins. The maximum land area requirement is for the land discharge alternative of approximately 1,200 net acres, assuming that additional land with favorable soil conditions and geology can be obtained. Alternative crops such as trees or greenchopped corn could reduce land area requirements slightly.

Nitrogen from effluent and biosolids should not cause adverse impacts to groundwater nitrate concentrations on fields where crops are grown. Additional nitrogen removal is recommended through either conventional means, wetlands, or operational practices to insure acceptable nitrate concentrations under percolation disposal fields or basins.

Costs for conventional treatment facilities associated with the alternatives are presented later in Section 11. The final evaluation of combined treatment and disposal/reuse alternatives is presented in Section 13.

REFERENCES

1 S. C. Reed et al. *Natural Systems for Wastewater Treatment.* Water Pollution Control Federation Manual of Practice FD-16, 1990.

2 Crites, R.W., Meyer, E.L., and R.G. Smith. *Process Design Manual—Land Treatment of Municipal Wastewater.* EPA 625/1-81-013. October 1981.

3 S.C. Reed et al. *Process Design Manual - Land Treatment of Municipal Wastewater—Supplement on Rapid Infiltration and Overland Flow.* October 1984.

4 Reed, S.C. and R.W. Crites. *Handbook of Land Treatment Systems for Industrial and Municipal Wastes.* Noyes Publications, 1984.

SECTION 11. TREATMENT PLANT UPGRADE AND EXPANSION ALTERNATIVES

The primary performance objective for the treatment plant processes is to achieve compliance with the waste discharge requirements at the design flows and loadings. Flow and loading projections were presented previously in Section 3. The current and anticipated discharge requirements were presented in Section 4.

This section describes alternatives developed for addressing deficiencies, increasing treatment plant capacity, and increasing treatment levels to serve the needs of the community through the year 2020 and beyond. In addition, this section presents general facilities improvements which are desirable because of safety, operational, or reliability considerations. This section focuses on the domestic wastewater treatment facilities. All cost estimates listed in this section include escalation to ENR 7000, a 40 percent contingency, plus engineering and administrative costs.

INTRODUCTION

The City of Lodi constructed the White Slough Water Pollution Control Plant in 1967 and expanded the treatment facilities in 1976 and 1990. Figure 11-1 presents a site plan of the existing treatment plant which provides comminution of industrial wastewater and complete treatment of domestic wastewater. Figure 11-2 presents a flow diagram of the overall treatment and disposal facilities for domestic and industrial wastewater. Figure 11-3 shows the locations of the ponds and treatment plant relative to roads and other nearby facilities. Appendix A contains a list of design criteria and additional facility data.

Existing Treatment Facilities

The existing domestic treatment process consists of grinding, grit removal, primary sedimentation, conventional activated sludge secondary treatment, and chlorine gas disinfection. Primary and secondary solids are further treated in anaerobic digesters and sludge lagoons. Treated effluent is used for agricultural irrigation of animal feed crops, as process water for the Northern California Power Agency power generation facilities, and for the Mosquito Abatement District pond feed water. After the crop irrigation season, treated effluent is discharged to surface waters. Effluent storage ponds are used to store treated domestic wastewater for irrigation purposes or to store untreated wastewater in an emergency.

Industrial wastewater passes through grinders within the domestic treatment plant headworks prior to lift pumps. After grinding, industrial wastewater is direct land applied for agricultural irrigation of animal feed crops during the growing season. The headworks facility also provides flow measurement and pumping. Industrial treatment and reuse are evaluated in Section 8 and discussed in this section only to the extent that common facilities are provided within the domestic treatment facilities.

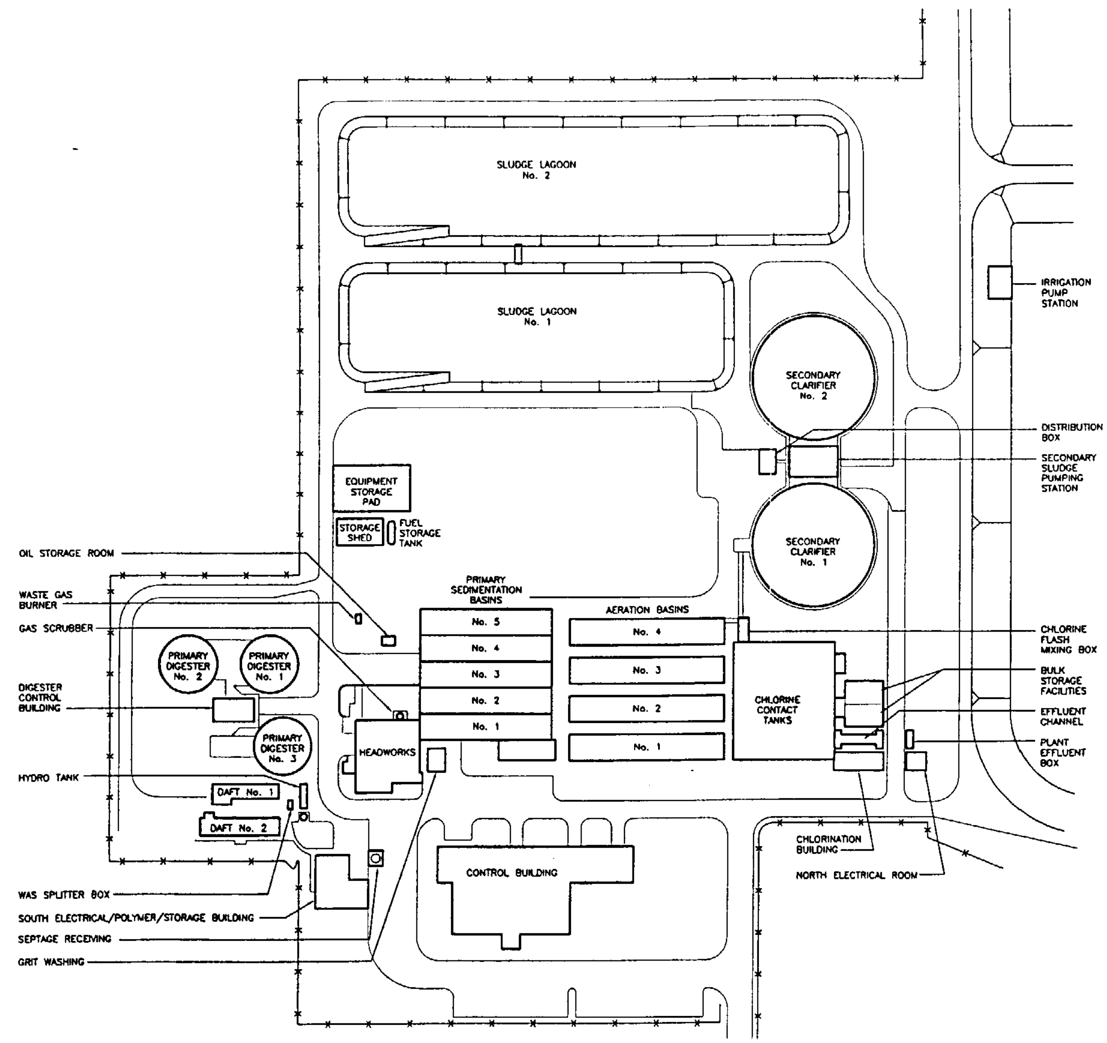


Figure 11-1

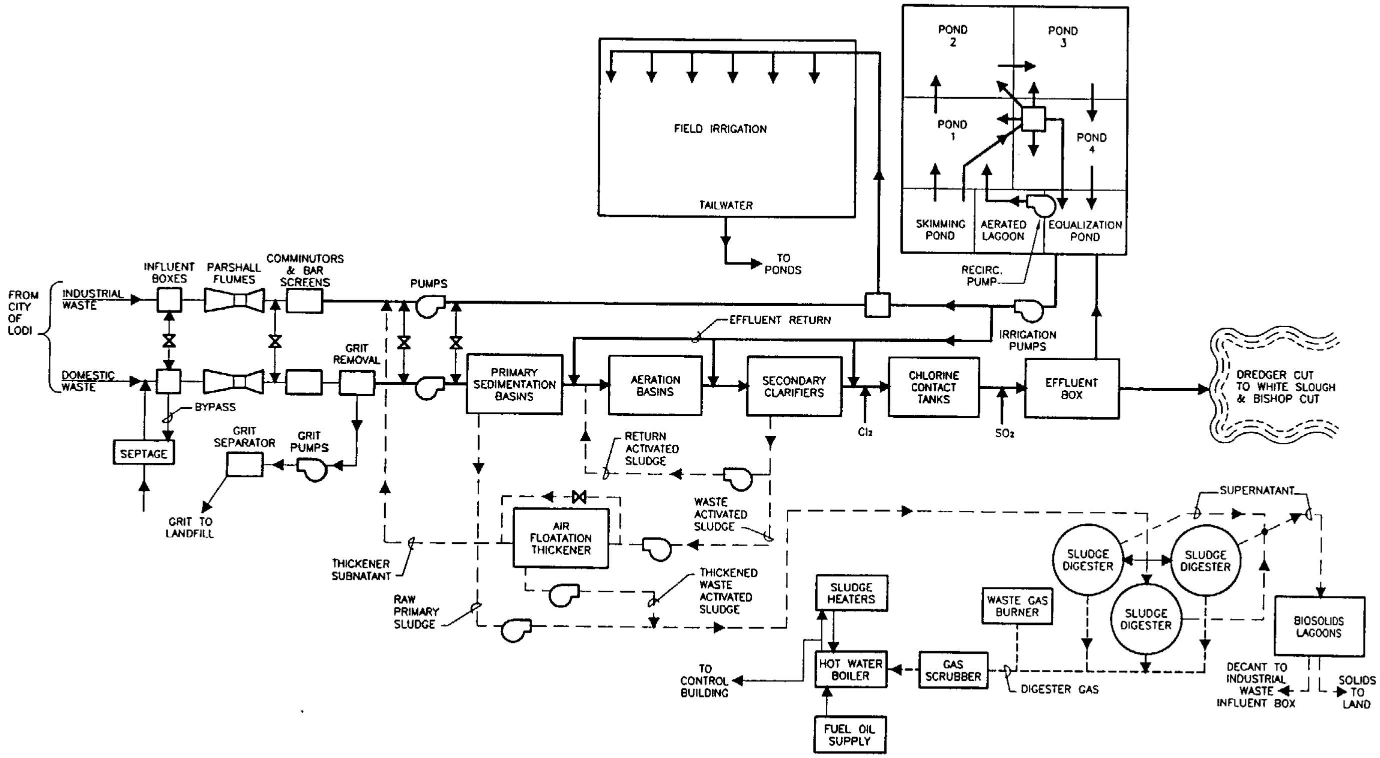
City of Lodi
Water Pollution Control Facility
Existing Site Plan

WEST YOST & ASSOCIATES

Figure 11-2

City of Lodi
Water Pollution Control Facility

Existing Treatment Plant Flow Schematic



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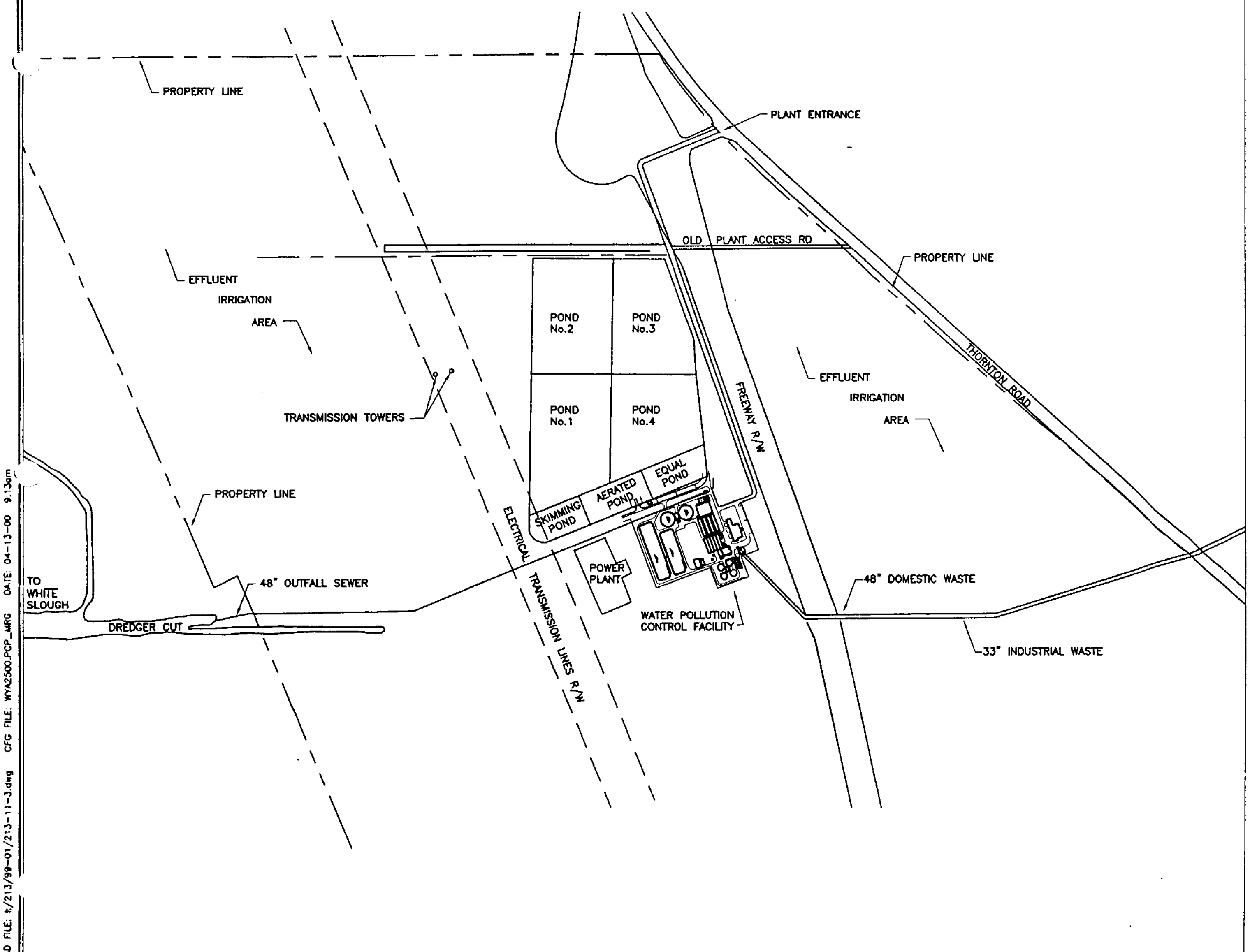

Figure 11-3

City of Lodi
Water Pollution Control Facility

Location of Treatment Plant & Ponds

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Facility Review Procedures

Process unit treatment performances were determined by reviewing historical operational records, compliance reporting, and past studies and reports. This performance was compared with industry performance standards. Preliminary process models were used where applicable to estimate reliable capacities of existing treatment processes for current, anticipated, and potential future waste discharge requirements. Based on these analyses and input from plant staff, capacities and operational deficiencies of plant process units were determined.

Rated Capacity

The capacity of the overall plant is currently limited by certain key processes to about 6.5 to 7.0 Mgd – substantially less than the 8.5 Mgd minimal capacity for the 1990 expansion. Since the last expansion, however, regulatory changes have required the plant to nitrify or remove ammonia. This requirement has effectively increased demands on the entire secondary process, requiring it to be rated at a lower capacity than the original basis for design. Additionally, several facilities which would have been included in the last expansion to provide redundancy and reliability were not constructed due to lack of funds at that time. The most notable areas lacking redundancy in the existing plant are the secondary clarifiers, dissolved air flotation thickeners for waste activated sludge, anaerobic digesters, and biosolids storage lagoons.

The following section presents a process evaluation and presents recommendations for "expanding" the plant capacity to 8.5 Mgd, to compensate for de-rating of process components to meet regulatory requirements while providing sufficient redundancy for outages of process components during required maintenance.

DESIGN CRITERIA FOR UPGRADES

General

Detailed unit process design criteria which control the sizing and capacity rating of treatment units are listed in Table 11-1. These criteria are based on generally accepted industry values, specific analysis of anticipated conditions and the experience of the facilities planning team. These criteria were used for sizing of unit process for all alternatives considered.

Treatment Requirements

Domestic treatment capacity has been affected by the need to meet more stringent discharge requirements than planned for when the last expansion was constructed. The purpose of the treatment plant review was to identify those improvements necessary to meet existing, anticipated, and potential future discharge requirements at Year 2020 projected flows and loadings. A related purpose of this review was to identify those improvements necessary for potential effluent reuse or disposal alternatives such as the proposed Sports Complex. The potential discharge requirements and alternatives for satisfying them were discussed previously in Sections 4 and 5. The major potential additions to the processes at the treatment plant are nitrification, nutrient removal, heavy metals removal, and tertiary filtration. For continued discharge to Dredger Cut, tertiary treatment and full nitrification are mandatory.

Table 11-1. Summary of Controlling Unit Process Design Criteria

Unit Process	Design Loading Condition	Process Criterion	Process Performance
Pretreatment			
Influent pumps	Instantaneous peak hour (Mgd)	Total pumping capacity at required head with one pump out of service.	Pump all flow under all operating conditions.
Mechanical screens	Instantaneous peak hour (Mgd)	One mechanical unit out of service and one fixed or mechanical screen as backup.	Screen all flow under all operating conditions.
Aerated grit removal system	Instantaneous peak hour (Mgd)	Detention time ≥ 3 minutes at peak hour with one unit out of service.	Treat all flow under all operating conditions.
Primary Treatment			
Primary sedimentation basins	Instantaneous peak hour (Mgd)	Surface overflow rate less than 2,500 gpd/ft^2 with one unit out of service. Preferred maximum overflow rate less than 2,000 gpd/sf for all units in service.	Average BOD_5 removal – 30%. Average suspended solids removal – 65%.
Aeration Basins			
Aeration Basins	Maximum month BOD and ammonia loading condition.	Design is based on nitrification with all units in service. Aeration requirements matched to peak BOD/ ammonia loads.	Soluble effluent BOD consistently below 15 mg/L. Ammonia consistently below 2 mg/L average and 7 mg/L peak.
Secondary Clarifiers			
Secondary Clarifiers	Peak sustained flow with all units in service.	Capacity rating based on Flux Analysis using: • SVI = 100 mL/g • RAS = 8.5 Mgd (total) • MLSS = 3,500 mg/L Maximum overflow rate less than 1,100 gpd/sf for all units in service.	Secondary Effluent BOD and SS consistently below 30 mg/L.

Unit Process	Design Loading Condition	Process Criterion	Process Performance
Disinfection System			
Chlorination System	Peak Hour Flow	Chlorine dosage of 20 mg/L for planning purposes.	Effluent coliform 2.2 MPN/100 mL.
Chlorine Contact Basins (advanced secondary)	Peak Hour Flow	30 minute contact time.	Effluent coliform 2.2 MPN/100 mL.
Chlorine Contact Basins (tertiary)	Peak Dry Weather Flow	90 minute modal contact time C*T > 450 mg-min/L.	Effluent coliform 2.2 MPN/100 mL.
Dechlorination System	Peak Hour Flow	Chlorine Residual 8 mg/L for planning purposes. Feed Rate = 1.46 lb sodium Bisulfite per lb chlorine residual.	Effluent chlorine residual < 0.01 mg/L.
Solids Processing			
DAFTs	Maximum month solids loading; largest unit in service.	Loading rate for waste activated sludge < 0.5 $lb/ft^2/hr$.	Thickened sludge concentration of 3.5%.
Digesters	Maximum month loading with all units in service. Average month loading with one unit out of service	Loading less than 0.15 lb $VS/ft^3/day$ and detention time greater than 15 days.	Class B Sludge. 50% Volatile Solids Destruction.
Digested Sludge Storage	Average digested solids loading.	Minimum 120 day storage.	Storage prior to land application without odors.
Tertiary Filtration (required for most discharge alternatives)			
Media Filtration	Peak Tertiary Flow	Max. 5 gpm/ft^2.	2 NTU max. daily average, 5 NTU max. 5% of time, 10 NTU max. peak.
Microfiltration	Peak Tertiary Flow	Varies by manufacturer.	0.2 NTU max. 5% of time, 0.5 NTU max. peak.

Nitrification. Ammonia removal through biological nitrification is required for all alternatives which include discharge to Dredger Cut or Bishop Cut.

Nutrient and Heavy Metals Removal. Additional treatment evaluated for the Master Plan includes nitrogen removal through biological denitrification, phosphorous removal through biological uptake, and additional heavy metal removal through chemical addition and enhanced effluent filtration. These alternatives are presented only to the extent that the Master Plan facilities provide flexibility to accommodate these and other facilities as they are required.

Tertiary Filtration and Disinfection. Tertiary filtration and disinfection of the secondary effluent are required for most of the surface waters discharge alternatives and for the reclamation of effluent for the Sports Complex or other unrestricted irrigation uses.

Redundancy

Process equipment should have sufficient redundancy or reserve capacity to handle incoming flows and continue to meet discharge requirements if one equipment item is out of service for maintenance or repair. For example, pump stations should have sufficient capacity to handle peak flows with one pump of the set out of service. Where units of different capacity are installed, the process should accommodate peak flow with one of the largest units out of service.

Staff and Public Safety Criteria

All applicable safety related codes and regulations must be satisfied when the treatment plant facilities are modified or upgraded. Safety regulations by Federal OSHA, Cal OSHA, and other regulatory agencies will be followed. Other safety related codes to be followed include the Fire Code and Uniform Building Code.

Americans With Disabilities Act (ADA)

The ADA and other related governmental requirements for handicapped access will need to be satisfied in the modification of existing buildings and construction of new buildings.

Flood Protection

100-year flood protection must continue to be provided for the treatment plant. This is a requirement in the waste discharge requirements and will probably be a requirement for any governmental loan or grant monies used for treatment plant upgrades.

Power and Control Redundancy

Electrical power backup and control redundancy will be provided in any upgrades or modifications to prevent spills or overflow of undisinfected effluent in the event of a power outage.

OPERATIONAL CRITERIA

Operational criteria refers to items related to operational strategies and philosophies rather than mandated design criteria. Operational criteria includes staffing levels, operational flexibility, control, and automation.

Staffing

The plant is currently fully staffed 5 days per week from 7:00 a.m. to 3:30 p.m. One operator is on duty from 3:30 p.m. until 11:00 p.m. weekdays and from 7:00 a.m. to 11:00 p.m. on weekends. This level of staffing has proved adequate in the past and should be adequate for the near future. As treatment processes are added and the treatment requirements become more stringent, plant operation may be increased to 24-hour staffing. The need for additional staffing will depend on the timing and extent of the improvements made with the next plant expansion or upgrade project.

Operational Flexibility

The current flexibility of plant operation should be maintained by the upgrade so that unit processes can be operated in different modes and with varying loading so as to optimize overall plant performance. Existing deficiencies which limit flexibility should be eliminated where feasible. Upgrades should also provide enough flexibility to allow for periodic maintenance of any unit process without undue disruption of the remaining processes necessary to achieve compliance with discharge requirements.

Automation Criteria

Existing control systems are limited to data recording and alarm of critical plant components. After hours alarm notification is made with an autodial to service, which calls out plant staff.

The existing system can be described as a programmable logic controller (PLC) system with main and local control panels. The PLC system consists of AB PLC 5/40 hot redundant standby processors inside the main control panel with remote PLC located in local control panel LCP-C which communicates over data highway cabling. Alarm, monitoring, and display functions are provided on the main control panel using conventional indicating lights, annunciator, vertical analog meters, and horizontal strip chart recorders. The PLC system as configured is SCADA ready, meaning that a SCADA graphic PC based system could be installed with minimum effort and expense now or during any phase of upgrades.

In general, the next plant improvement project should incorporate instrumentation and control features which increase plant automation if such automation will result in improvements in one or more of the following areas:

1. Safety
2. Treatment reliability
3. Reduced operator workload
4. Reduced operating cost

5. Convenient alarm notification
6. Offsite data access

As a minimum, new or updated process control equipment should be capable of automation as shown in Table 11-2.

Table 11-2. Plant Automation Criteria

Control Function	Recommended Level Of Automation
Aeration Blower Control	Full automatic control
Disinfection Chemical Dosing	Full automatic control
Dechlorination Chemical Dosing	Full automatic control
Digester Heating	Full automatic control
Influent Pumping	Capable of upgrade to full automatic control
Primary Sludge Pumping	Capable of upgrade to full automatic control
RAS/WAS Pumping	Full automatic control

Full automatic control implies computer control with redundant instrumentation. A fully automatic system can be started, stopped, and adjusted based on programmed process set points, time of day, or other pre-programmed criteria. Instrumentation must provide reliable control feedback and alarm capabilities. Automatic control provides two potential advantages. First, power and chemical use can be optimized through continuous monitoring and adjustments. Second, full plant automation can reduce operator workload so that more time can be devoted to preventative maintenance, or the need for 24-hour staffing can be delayed.

The recommended operational and control criteria that should be provided with any plant expansion are summarized in Table 11-3. Additional criteria details are discussed below.

Table 11-3. Summary of Operation and Control Design Criteria

Plant Equipment or Area	Design Criteria	Desired Results
Alarm Systems	PC based auto dialer with prioritization and individual addressing. Laptop dial-in to remotely access plant graphic screens	Better communication, information, and response
Equipment or Process Control	Local control, remote monitoring for all treatment equipment; full SCADA on major process control (Aeration Basin DO, RAS, WAS, Disinfection/ Dechlorination, Digester Heating)	Increased process monitoring and centralized control capabilities; improved automation capabilities; increased process efficiency to reduce power and chemical costs
Control Zone Loops	PLC Control	Simplicity, enable local or SCADA manual control

Plant Equipment or Area	Design Criteria	Desired Results
Control Zone Operator's Interface	Color graphics, on-screen	Quick, easy comprehension to shorten response time and reduce potential for errors
Zone Computers:		
CPU Hardware	High quality standard PC	Lower cost, ease of upgrade and change ability compared to conventional hardwired control panels
Display	PC Screen	Lower cost, ease of upgrade
Power System	15 to 30 minute uninterruptible power supply with conditioning	Reliability
Supervisory Control and Data Acquisition Software	PC Based	Availability of add-ons or upgrades, ease of programming, flexible data management and reporting to improve analytical capabilities and reduce workload
	Third party SCADA software: Wonderware, Intellution or similar.	Setup and support available from many sources.
	Allen Bradley communication protocol	Compatibility with existing City systems
PLCs	Allen-Bradley with Ethernet connection to PCs.	Compatibility with existing equipment, similarity of replacement parts, leverage staff knowledge and reduce training costs
Communication Link	Fiber Optics	Noise immunity and ground loop isolation

Control and Data Acquisition

Equipment and processes at the plant could be divided into control zones linked by a plant-wide supervisory control and data acquisition (SCADA) system. All control and instrumentation for any one process can be tied to a single zone; however, multiple processes may be controlled by a single zone for plant redundancy. The control zone concept could extend to auxiliary systems such as plant water, intrusion monitoring system, HVAC, *etc.*

Each zone control center may consist of a programmable logic controller (PLC), and a networked zone computer with a standard color graphic CRT display. Motor control centers should have few or no switches and indicator lights; all remote control may be operated via the zone computer, or over the SCADA system from another zone control computer. Use of local control panels will be minimized. Each computer should have a standard CRT display and mouse which will serve as the operator interface.

Manual override control of equipment should be provided through local switches and speed control (where appropriate) located adjacent to the equipment. These local control stations will include limited digital indicators providing the minimum necessary status information.

Controls should be standardized as much as possible throughout the plant and should be easily adjustable or programmable by treatment plant staff. Communications and PLC hardware will be based on the Allen Bradley specifications for compatibility with existing control equipment. Any control system should provide adequate control redundancy through dedicated analog indicators and indicating lights in each control zone, or redundant PLC should be provided for each control zone.

Data acquisition should be performed at a central computer in the operations building receiving signals from each of the control zones by way of the central PLC. The existing main control panel may no longer be necessary with existing functions such as trending, status and alarm indication done in graphics. The data acquisition hardware should utilize widely accepted industry standards and be non-proprietary to the greatest extent possible. The SCADA software should be graphical, easy to use, and based on a non-proprietary operating system with multi-sources of support. Operator training must be available and similarity to the existing data acquisition software will reduce training and start-up costs.

PRELIMINARY TREATMENT

The Water Pollution Control Plant headworks facility provides preliminary treatment consisting of flow measurement, comminuting, grit removal, and pumping.

Description of Existing Facilities

The headworks facility is a below grade structure. The pump room sets below an electric panel room and a sludge heater room. Inlet air blowers and exhaust fans provide fresh air and purge odors. The air blowers and exhaust were recently upgraded. Exhaust air is discharged at the headworks roof. No odor scrubbing is provided.

Raw wastewater enters the plant headworks influent boxes from the east in a 48-inch domestic waste sewer and a 33-inch industrial waste sewer. Hydraulically operated sluice gates at the influent box automatically close upon a plant power failure preventing wastewater from entering the plant. Upon gate closure raw sewage backs up into the influent sewers. Raw sewage may then be diverted around the treatment plant and dumped into the effluent equalization pond. However, the equalization pond water level must be lowered sufficiently to allow for this plant bypass to occur.

The domestic and industrial waste streams are treated separately through the headworks. The domestic waste flow is measured in an 18-inch Parshall flume (expandable to 24-inch), passed through a comminutor, treated for grit removal, and pumped to the domestic wastewater treatment facilities. The industrial waste flow is measured in a 12-inch Parshall flume (expandable to 18-inch), passed through a comminutor, and pumped directly to land for crop irrigation and/or direct land treatment. The existing Parshall flumes are adequate for projected flows within the Master Plan planning period.

Septage is dumped into the domestic sewer upstream of the headworks. A septage receiving station and 4,000-gallon holding tank (added in 1975) are not used because the holding tank has proved to be too small, and the holding tank piping is subject to plugging. An estimated 720,000 gallons of septage was received in 1998. Grease from local haulers within the City limits is dumped into the primary scum pits and pumped directly into the anaerobic digesters. An estimated 230,000 gallons of grease was received in 1998.

The domestic wastewater grit removal system consists of two 16-foot diameter settling tanks referred to as a detritus tanks. Collected grit is pumped through a cyclone/classifier grit dewatering system and then deposited into a dumpster and periodically hauled to a landfill. As much as 15 cubic yards of grit is removed every 2 to 3 weeks.

Influent pumps consist of three domestic pumps and two industrial pumps. Pumps are powered by variable frequency drives. The drives are outdated and difficult to repair. Plant staff have begun a program to replace all variable frequency drives with new equipment. Each influent pump is fed from an individual 4-foot by 7-foot wet well. Flow is directed to the wet wells through several manually operated slide gates from a single cross channel. All pumps discharge to a pump discharge channel ahead of the primary sedimentation basins. The industrial waste discharge line begins at the pump discharge channel. Flow is directed to the industrial discharge line or to the primary sedimentation basins through several manually operated slide gates.

Deficiencies and Alternative Improvements

The headworks is in need of rehabilitation. Existing equipment is outdated and parts of the structure are showing significant corrosion. Except for the influent pumps, all existing headworks components have sufficient capacity through the Master Plan planning period. Expansion will be required to handle flows beyond 8.5 Mgd.

Headworks Structure. The existing headworks structure is a below grade structure that is difficult to expand or modify to accommodate improved equipment and higher capacities. In general the headworks concrete surfaces are partially deteriorated at floor level and in the wastewater channels. The concrete at the floor of the headworks structure appears to be superficially corroded. Concrete in the flow channels has experienced corrosion and has required repairs in the past. All concrete surfaces should be cleaned, inspected, and refinished as required. Existing channel grates which deflect and move under foot should be replaced with new aluminum grating. The alternative of constructing a new headworks is discussed later in this sub-section.

Control Gates. The hydraulic pump for powering hydraulic equipment is located in a deep pit making access difficult and should be relocated. Most of the manually operated slide gates at the cross channel ahead of the pump wet wells and the slide gates in the pump discharge channel are not operable. The slide gates should be replaced and/or rehabilitated as required to restore full operation. Gate operators should be added to facilitate operation of critical slide gates that need to be operated in an emergency condition such as the gates at the cross channel ahead of the pump wet wells. These gates may be used to direct industrial flow to one of the domestic pumps during failure of an industrial influent pump.

Removal of Large Solids. Comminuting is not in wide use for wastewater treatment. Comminuting grinds rags and other debris into smaller pieces but does not remove this objectionable material which is clogging downstream treatment systems and filling the digesters with debris. The preferred approach is to remove the rags and other coarse material from the flow stream with self-cleaning screens.

There are three general types of screens that can be used for the influent screening application:

1. Mechanically cleaned bar screens
2. Continuous self-cleaning bar/filter screens
3. Spiral basket screens

Mechanically cleaned bar screens consist of fixed bars in a channel that collect debris and a motorized rake mechanism that cleans the debris off the front of the screen, and lifts the debris to a discharge point some distance above the channel. The only moving part of the screen that is submerged is the rake. Theses units require high overhead clearance and are not suitable for fine screening.

Continuous self-cleaning bar/filter screens consist of a continuous belt of plastic or stainless steel mounted horizontally and vertically on a series of parallel shafts that collect, convey, and discharge all solids greater than a mesh size from ¼ inch to 3/4 inch. This type of screen is often selected over mechanically cleaned bar screens where greater capture is required and high continuous loading is expected. Collected screenings can be washed, dewatered, and compacted in separate units. Continuous self-cleaning bar/filter screens do not need to operate 24 hours per day. Adjustable timers are used to match daily flow variations. Selection of the screening system should be based to a great degree upon the ability of the system the minimize head loss and reduce impacts to the Parshall flume. As a general rule, the maximum allowable flume submergence for a flume one foot wide or greater is 70 percent (ratio of downstream flow depth to upstream flow depth) before free flow conditions are interrupted and flow measurement is hindered. Preliminary review of the headworks hydraulic profile and head loss data provided by one screen supplier indicates that new screens may be inserted into the 32-inch communitor channels with minimal impact to Parshall flume operation. This is based on no more than a 30 percent blocked screen and an approach velocity of about 2 feet per second. Some minor modifications to the channel bottoms and walls would be required.

Actual screen head loss data for continuous filter screens should be collected at design flow conditions at a similar facility before screens are installed at Lodi. However, one screen would not treat all flow during peak flow conditions should a screen be out of service. It would be necessary to provide an overflow weir to the existing bypass channel to prevent Parshall flume submergence during high flow conditions with one screen out of service. This would still be a significant improvement over existing conditions.

Spiral basket screens consist of a ¼-inch screen basket and brush located in the flow channel, screenings washing agitator, shaftless spiral conveyor assembly, transport tube, and discharge chute. Spiral screens combine screening, screenings washing, dewatering, and compaction into one unit. The spiral screens are compact and ideally suited for low flows. Their primary disadvantage is that their rated capacity is limited to 7 Mgd each. Two units would have a maximum theoretical capacity of 14 Mgd at peak flow. The actual capacity will vary with the quantity of screenings and amount of basket plugging. With severe plugging the units will back up the water surface into the Parshall flume and overflow the top of the basket screen. The existing bypass channel could be fitted with a third screening unit or an overflow weir could be placed in the bypass channel to prevent Parshall flume submergence during high flow conditions or basket plugging. In this case a portion of the flow would not be screened. The other disadvantage of the basket screens is that the washing mechanisms at the bottom of the channel are not effective at water depths greater than 22 to 24 inches. As the flows increase and the screens begin to plug the water depth will rise. This will lead to diminished washing effectiveness, more plugging, and deterioration of screenings quality. For this reason it is important that the units not be overloaded. Actual screen headloss data for basket screens should be collected at flow conditions near 7 Mgd at a similar facility before screens are installed at Lodi. In addition, alternative designs recently on the market should be reviewed where the screen capacity is not limited to 7 Mgd.

Continuous self cleaning bar screens are the recommended alternative at this time. The cost of self-cleaning screens is similar to spiral basket screens and only slightly greater than for mechanical bar screens. Spiral basket screens are not recommended because their capacity would be limited given the existing channel geometry. Spiral basket screens are still evolving rapidly and should be reevaluated during the predesign phase of the next treatment plant upgrade. The use of mechanically cleaned bar screens is not recommended because of the overhead clearance requirement and the fact that mechanically cleaned bar screens are not suitable for fine screening.

The cost of two new continuous, self-cleaning screens including installation, channel modifications, and electrical power is estimated at about $500,000. The total cost with a screw conveyor system is estimated at about $600,000. The actual cost will depend on the final screen selection and the number of screens purchased.

Compacted screenings could be discharged to garbage barrels or small dumpsters and lifted out of the headworks each day through a roof access hatch. Alternatively, a hydraulic ram press or a screw conveyor could be used to lift the compacted screenings out of the headworks through a new roof opening and deposit the material directly into a dumpster located at ground level.

Grit Removal. Detritus tanks are not in wide use for wastewater treatment. These tanks do not provide consistent grit removal during varying plant flow conditions. At low flows excessive organic material is removed and during high flows grit is washed downstream. However, the grit washing and dewatering system is designed to remove excessive organics which overcomes some of the deficiencies of the existing grit removal system. The existing grit removal hopper and piping is subject to plugging during peak inflow events or during a pump failure. The space for grit removal pumps and piping is very limited and makes pump repair work difficult and there are no redundant pumping units to operate during a pump failure. The existing grit removal tanks are not amendable to modification or expansion due to the confined space within the headworks.

Aerated grit tanks are relatively small rectangular structures with steep-walled grit hoppers. Coarse-bubble aeration creates a rolling action in the tank, which keeps lighter organic material in suspension. Screw augers or recessed impeller pumps remove the collected grit from the grit hopper. Aerated grit tanks offer the added benefit of preaeration of the raw sewage. Preaeration can help prevent septic conditions in the primary clarifiers and remove corrosive or odor-causing compounds.

A vortex grit removal system consists of relatively small circular structures with a center grit collection hopper. Incoming wastewater and small propeller blades above the grit hopper produce a spiraling vortex action that tends to lift lighter organic particles. Recessed impeller pumps remove the grit for further washing and dewatering. Vortex grit removal systems use less energy than aerated grit tanks. The principle disadvantages of this system are relatively greater potential for grit collection hopper plugging and the fact that the wastewater does not receive any aeration prior to primary treatment.

New domestic wastewater grit removal facilities are recommended to replace the existing grit tanks. The aerated grit removal system is the preferred alternative at this time. A new grit removal system will require construction of new facilities outside the existing headworks. The most likely site for new facilities is the open area located to the southwest of the existing headworks. The existing oil storage room and waste gas burner would be relocated to allow room for the new facilities. Grit removal would occur after influent pumping so that the grit removal facilities could be built at a higher elevation for lower cost and better accessibility. Existing pump discharge piping would be extended to the new grit removal facilities. After grit removal, all flow would be returned to the existing pump discharge channel ahead of the primary sedimentation basins. The estimated construction cost of a new grit removal facility is over one million dollars. This expenditure of funds is not justifiable within the Master Plan planning period. However, in the long-term the grit removal system should be replaced with a new structure because it will become impractical to expand the existing structure when average flows exceed 8.5 Mgd. The need for new grit removal facilities is far enough into the future that vortex grit removal systems should be reconsidered prior to design.

Septage. The existing septage receiving station consists of a removal lid centered in a small wash down area where waste haulers park and discharge their truck contents into a small holding tank. The tank contents can be discharged to either the industrial or domestic influent sewers. The existing septage holding tank can only hold the contents of one large septage hauling truck. When there is more than one truck, the holding tank is not effective. The existing station is not used because the holding tank has proved to be too small to be of much value and the holding tank piping is subject to plugging. As a result, the holding tank is out of service and waste haulers dump their waste directly into a manhole on the domestic influent sewer adjacent to the septage receiving station. This waste can sometimes contain large debris that can cause problems in the treatment plant headworks.

Undiluted septage dumps are undesirable. The rapid dumping of 3,000 gallons or more of odorous septage into the plant influent flow stream creates a shock odor condition. Additional septage storage capacity is required. A new septage receiving station could be installed which provides storage for holding the peak daily volume of septage. The required tank storage volume is estimated at 20,000 gallons total with two tanks based on as many as 8 trucks a day with a septage volume of 2,000 to 3,000 gallons per truck. The station should be equipped with a large, hard-surfaced wash down area, wash down water, coarse bar screening, spray water, and foul air withdrawal and odor scrubbing. Facility design should include the ability to pump septage directly to the digesters. Additional features such as water flushing system inside the storage tank, flow metering, pH monitoring, or controlled access through a card-lock system could also be provided. A separate receiving station will require construction of new facilities outside the existing fenced treatment plant site. The most likely site for new facilities is the open area located to the east of the existing South Electrical/Polymer/Storage Building.

One alternative would be to construct a new septage receiving facility at an estimated cost of approximately $400,000. A second alternative would be to reconstruct the existing septage receiving facilities to provide a hard-surfaced wash down area, coarse bar screen, and an unobstructed discharge to the domestic influent sewer. The cost of reconstruction of the septage receiving facility is estimated at approximately $60,000.

New septage facilities are ultimately needed, but may be delayed for several years while sufficient funds are accumulated for the new facilities. Plant performance impacts do not justify such an expenditure of funds within the Master Plan Planning Period. New septage facilities should be constructed when a new headworks is constructed in the next expansion after the end of the Master Plan planning period.

Influent Pumps. Influent pump capacity is inadequate to meet projected peak hour demand of 16.3 Mgd with one unit out of service. All domestic pumps will be required to operate during projected peak flow conditions, and all industrial pumps are required to operate during peak flow conditions. The existing cross channel in front of the individual pump wet wells allows pumps to be used interchangeably for domestic or industrial waste. However, most of the manually operated slide gates at the cross channel ahead of the pump wet wells and the slide gates in the pump discharge channel are not operable. In addition, it would be very awkward to make the channel and valve changes necessary to switch the industrial pumps to handle domestic wastewater during peak conditions. Therefore, the gates and channels should be replaced and/or rehabilitated. The cross channel would only be used for emergency operation.

Domestic influent pump capacity should be increased to pump peak flows with the largest unit out of service. The two existing pumps (7.2 Mgd, 50 hp) should either be sped up or replaced with larger pumps with a capacity of about 5,900 gallons per minute each (8.5 Mgd, 60 hp).

Industrial influent pump capacity should be increased to pump peak flows with the largest unit out of service. The problem with only increasing the capacity of the existing pumps is that during the non-canning season the pumps would be too large and would be cycling off and on too frequently. Therefore a third industrial influent pump should be installed to handle the non-canning season flows. The smaller of the two existing two pumps (4.7 Mgd, 30 hp) should be replaced with a pump which has a capacity of 7.2 Mgd to match the existing larger pump. Although this would not provide full redundancy, it would handle the peak day industrial flow assuming peak hour flow could be allowed to back up into the sewer trunk. One of the domestic wastewater pumps can be also used as additional backup for the industrial pumps through the use of the cross-channel gates.

The third industrial wastewater pump should have a variable speed drive and a capacity of approximately 300 gpm (0.43 Mgd). This pump could operate at low speed to pump non-canning season industrial flows. The existing influent pump room is not designed for a third full sized industrial influent pump. A small pump could probably be installed south of Industrial Waste Pump No. 1. It would need a separate suction pipe installed into the wet well through the separating wall. It would discharge into the Industrial Waste Pump No. 1 discharge piping. A new check valve would need to be installed on the Industrial Waste Pump No. 1 discharge line to prevent backflow.

New Headworks. A new headworks could be constructed either in the next few years or around Year 2020 when the capacity of the existing headworks is close to being exceeded.

New headworks would include new channels, flow meters, screens, and influent pumps. The influent pumps would be located ahead of the other headworks processes in order to raise the headworks out of the ground. This would allow gravity flow through the headworks to the grit removal tanks and primary clarifiers. Grit removal tanks would be located immediately downstream of the screens. The location and general layout of new headworks are shown in Figure 11-4.

The cost of a new headworks structure with a new influent pump structure, flow measurement, continuously cleaned screens, and aerated grit removal is estimated at about $4,000,000. This is about double the estimated cost of modifying the existing headworks. The benefit in plant operation or process performance is not sufficient to justify the cost of a new headworks at flows up to the capacity of the existing headworks. However, expenditures made in the existing headworks will ultimately be thrown away, as a new facility will ultimately be required. The headworks must be replaced with a new structure when the treatment plant capacity is increased beyond 8.5 Mgd because it will be impractical to expand the existing structure. Alternatively, the City may opt to replace the headworks now to provide facilities which would be suitable for the next several decades.

Odor Control

The White Slough Water Pollution Control Facility has no odor control facilities. Odor control will become more of an issue as proposed facilities near the treatment plant are developed. The headworks and preliminary treatment facilities are significant sources of odor. Odor control measures are described and evaluated later in Section 12.

Summary of Recommended Facilities

Recommendations for new facilities associated with preliminary treatment are summarized as follows:

- Rehabilitate the headworks structure. All concrete surfaces should be cleaned, inspected, and refinished as required. Replace deteriorated grates with new aluminum grating. Relocate the hydraulic pump for powering hydraulic equipment. Replaced and/or rehabilitate all channel gates as required to restore full operation and add gate operators to facilitate operation of critical gates.

- Replace the existing domestic comminutors with self-cleaning screens at the next treatment plant upgrade. A continuous cleaning bar screen appears to be the best alternative at this time, but final selection of a screen type or manufacturer should be made during predesign based on headloss and operating data from units operating

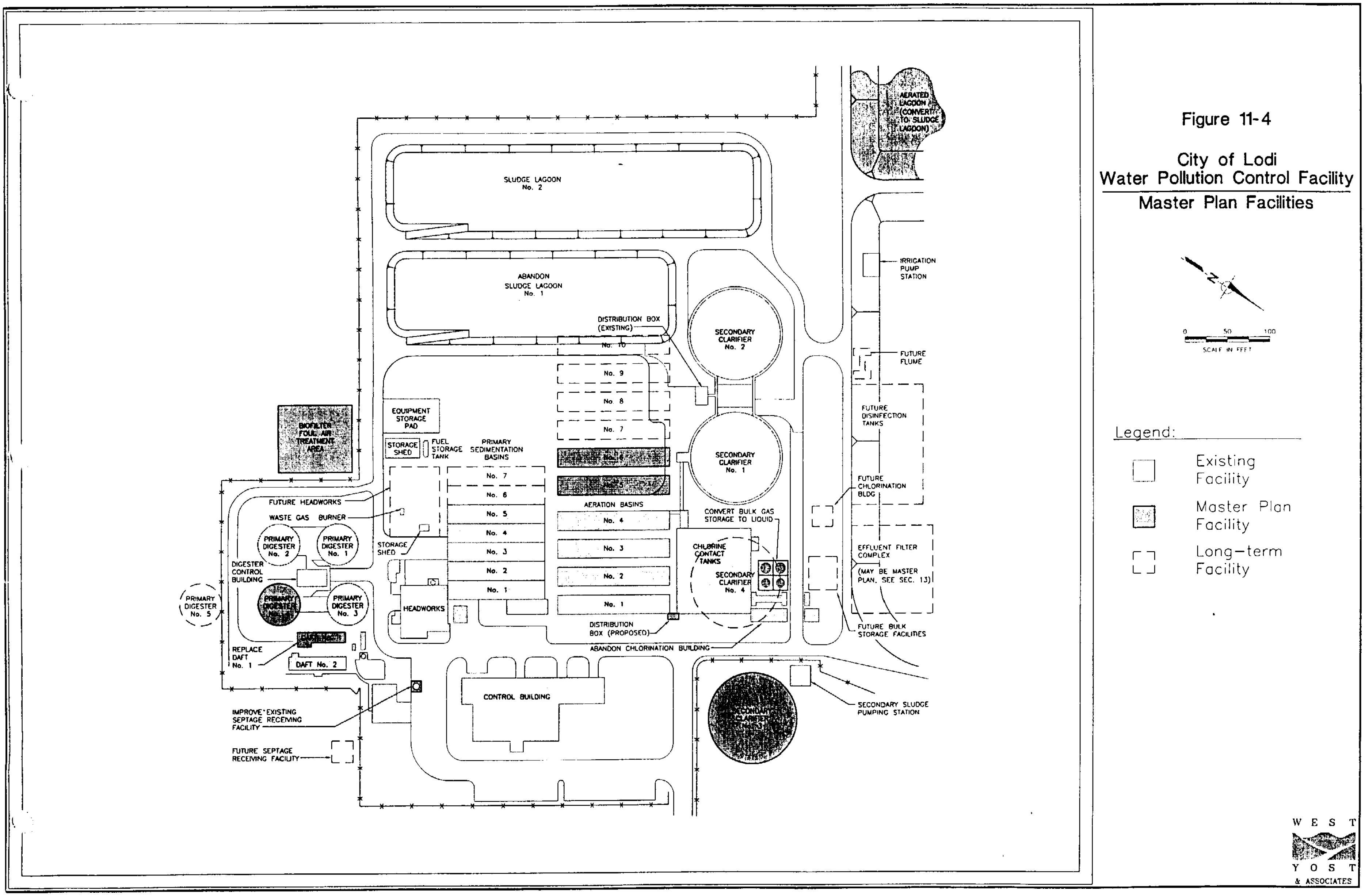

Figure 11-4

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under similar conditions. The self-cleaning screens should provide screening washing and compaction. This recommendation does not apply to industrial wastewater. There is no significant advantage to screening the industrial wastewater. Therefore, retain the existing industrial comminutors and or replace them with new grinders when the existing units are no longer serviceable.

- When treatment plant capacity is expanded beyond 8.5 Mgd, the detritus tanks should be replaced with new aerated grit tanks. The open area located to the southwest of the existing headworks should be retained for future grit removal facilities. No action is required during the Master Plan planning period.

- During the next treatment plant upgrade the existing septage receiving station should be modified to provide a hard-surfaced wash down area, coarse bar screens, and a discharge to the domestic influent sewer. After these modifications, the existing septage receiving station should be restored to full operating status. Within approximately 10 to 15 years, new septage facilities should be constructed in the open area located to the east of the existing South Electrical/Polymer/Storage Building.

- Replace the smaller existing domestic influent pumps or increase their capacities to 5,900 gallons per minute each (8.5 Mgd).

- Replace the smaller existing industrial influent pumps with a larger pump with a capacity of at least 5,000 gallons per minute each (7.2 Mgd). Install a third pump with a capacity of approximately 2.5 Mgd for non-canning season pumping and peak period backup capacity.

- When treatment plant capacity is expanded beyond 8.5 Mgd, the headworks should be replaced with a new structure.

- As an alternative, the City may wish to invest in a new headworks facility that will serve beyond the 8.5 Mgd master plan design capacity.

PRIMARY TREATMENT

There are five primary clarifiers (sedimentation basins). The five basins remove readily settleable solids and floating material from the flow stream, constituting primary treatment.

Description of Existing Facilities

The primary sedimentation basins are 90 feet long, 20 feet wide, and 8 feet deep. The basins are located immediately north of the headworks as shown in Figure 11-1. Settleable sludge accumulates at the bottom of the basins and is then removed via a chain and flight system and sludge pumps. City staff have replaced all chain and flight systems with plastic components. Plastic has proved to be less durable than metal components, but plastic is much easier to service and repair. The sludge is pumped to the primary digesters periodically, which means that sludge is stored and thickened in the bottom of the basin. Grease, oil, scum, and other floatable materials are skimmed from the water surface by water sprays and helical skimmers. The skimmed materials (scum) are pumped directly to the primary digesters. Effluent flows over V-notch weirs at the end of each basin. Primary effluent flow from all basins is combined in an aerated channel which delivers flow to the aeration basins.

The primary sludge pump room houses ten progressive cavity pumps, five for primary sludge and five for scum. Pumps are direct driven through a belt and pulley system. Mechanical variable speed drive units originally used for sludge pumping before 1989 have been replaced with direct drives. Sludge pumps added in 1990 with the construction of primary sedimentation basins No. 4 and 5 have proved to be over-sized with capacities as much as 175 gallons per minute, but staff are able to use all units by reducing the operating time. The older sludge pumps have an estimated capacity of 80 to 120 gallons per minute. In the last few years, City staff have replaced all old sludge pumps installed before 1990.

Performance

Primary clarifiers should remove 60 to 70 percent of the influent total suspended solids (TSS) and 25 to 40 percent of the influent biochemical oxygen demand (BOD_5). Table 11-4 presents the combined primary clarifier performance for percent removal of TSS and BOD_5. The removal percentages are monthly average values. Based on historical data, the performance of the existing primary clarifiers has averaged 29 percent BOD_5 removal and 65 percent TSS removal.

Table 11-4. Primary Sedimentation Performance

Parameter	Range	Average
BOD		
Influent, mg/l	232 to 324	272
Effluent, mg/l	163 to 226	192
Removal, %	17 to 45	29
TSS		
Influent, mg/l	199 to 331	244
Effluent, mg/l	61 to 118	85
Removal, %	51 to 75	65

Note: Based on monthly averages, Aug 1994 to Jan 1999.

Design detention time of 1.5 hours and peak hour surface loading rates of about 1,800 gallons per day per square foot at 8.5 Mgd are within unit process design criteria previously presented in Table 11-1. The primary clarifiers are, therefore, rated with a firm capacity of at least 8.5 Mgd based on average flow conditions. The existing primary clarifiers have adequate capacity for the Master Plan planning period with no modifications. Additional primary clarifiers will be needed when flows exceed approximately 9.5 Mgd ADWF.

Summary of Recommended Facilities

The existing primary sedimentation basins are adequate with no modification for projected flows within the Master Plan planning period. The open area to the west of the existing basins (see Figure 11-4) should be retained for future expansion.

SECONDARY TREATMENT

Secondary treatment is provided through an aerobic suspended-growth biological system consisting of aeration basins and secondary clarifiers. The aeration basins provide a rich environment for accelerated biological growth which converts biodegradable, organic waste material into cell mass which can be removed through settling. Mixed liquor flows from the aeration basins to the secondary clarifiers where the solids are settled out of the wastewater stream. The settled solids are either pumped back to the aeration basins as return activated sludge (RAS), or to the sludge thickeners as waste activated sludge (WAS).

Description of Existing Facilities

Aeration Basins. Each of the four aeration basins is 137 feet long by 30 feet wide and 15 feet deep. The basins are located immediately north of the primary clarifiers as shown in Figure 11-1. At a water depth of 15 feet, the volume of each basin is about 0.46 million gallons. Primary effluent flows from the primary sedimentation basins to the north end of Aeration Basin 4 (the western most aeration basin). At this point return activated sludge (RAS) from the secondary clarifiers discharges into the primary effluent flow stream as it enters the aeration basin. The aeration basins are operated in series (4-3-2-1), and thus the mixed liquor flows through each basin. From the north end of Aeration Basin 1, the mixed liquor leaves the aeration basins through a 60-inch pipe to the secondary clarifier influent distribution box. The basins are provided with alternative primary effluent and RAS feed points and may be operated in series, parallel or divided into two independent aeration system.

Aeration is supplied to the mixed liquor through a series of headers with membrane tube diffusers. The air supply provides the required oxygen and keeps the aeration basins mixed. The existing diffusers are evenly distributed between four basins. Each basin contains five headers which feed 120 diffusers for a total of 600 diffusers per basin. The original Wyss Flex-A-Tube diffusers in three basins were recently retrofitted with FlexLine fine bubble diffuser membranes by EnviroQuip. The FlexLine diffuser membranes are reported to have a higher efficiency and are considered to be more durable than the Wyss diffuser membranes.

The air supply has normally been turned off at the first two diffuser headers at the north end of Aeration Basin 4 where primary effluent and RAS first enter the aeration basins. The intent has been to create an unbaffled anaerobic zone within the first 40 percent of the aeration basin and thereby create conditions favorable for improved settleability of the mixed liquor.

The secondary treatment system is normally operated to provide full nitrification to prevent ammonia toxicity in the effluent. In order to maintain nitrification, the mixed liquor suspended solid concentration is maintained at 3,000 mg/l or greater to achieve a sludge age of 6 to 8 days. During the Winter months the sludge age must be increased to as much as 10 days to maintain nitrification because of cooler temperatures and slower reaction rates. As the mixed liquor suspended solids concentration is increased, the aeration system is unable to maintain dissolved oxygen levels consistently through out the basin. Under such operating conditions, filamentous organisms may predominate which leads to foaming on the surface of the aeration basins and effluent channels. Foaming in such cases is controlled through the use of chlorine solution surface sprays.

The purpose of the aeration basins is to convert the waste into biological cell mass, which can later be settled out of the water in the secondary clarifiers. The concentration of soluble BOD_5 and ammonia in the secondary clarifier effluent is a measure of the performance of the aeration basins. Effluent ammonia levels are maintained below 2 mg/l under most operating conditions. Effluent BOD_5 levels are generally maintained below 10 mg/l on a monthly average condition. The existing aeration basins are not capable of reliably maintaining these treatment levels during periods of peak demand in Winter months.

Secondary Clarifiers. Each of the two circular clarifiers is 100 feet in diameter with a side water depth of 16 ft. The total volume of the basins is about 1,880,000 gallons, and the total surface area is 15,700 ft^2. Secondary Clarifiers 1 and 2 are identical, incorporating a flocculator center well, sludge collection mechanism, scum skimmer, and an outside weir and effluent launder. Chlorine solution is applied near the overflow weirs to control algal growth. It is important to note that the need to consistently achieve full nitrification requires that the concentration of solids in the mixed liquor from the aeration basins (MLSS) is relatively high. The MLSS concentration as flow enters the clarifiers is between 3,500 and 4,500 mg/L during late Fall and Winter months. This concentration is much higher than would be necessary if full nitrification were not required, and is outside the original design parameters of the two clarifiers.

The RAS and WAS pumping system is located in the Secondary Sludge Pumping Station located between the two clarifiers. RAS pumping consists of three 75 hp pumps with a capacity of 3,000 gpm @ 57 ft TDH each. WAS/Scum pumping consists of four 5 hp pumps with a capacity of 160 gpm @ 39 ft TDH each. Two pumps are dedicated for WAS pumping and two pumps are dedicated for scum pumping. Pumps can be interchanged between WAS and scum pumping as needed. All RAS and WAS pumps are equipped with variable frequency drives.

Air Supply. The aeration basins are supplied air from four 150 hp multiple-stage centrifugal blowers located in the Control Building Blower Room. The blowers are rated at 3,000 standard cubic feet per minute (scfm) each. Air is delivered through piping ranging from 6 to 24 inches in diameter to the 5 headers for each basin which feed the diffusers.

The Control Building Blower Room houses the four aeration basin blowers and a 15 hp blower for channel aeration. The channel aeration system is out of operation because the aeration systems in the flow channels need rehabilitation. The Blower Room is designed for a total of six aeration basin blowers. The existing blower inlet header is 30-inch. The existing blower outlet header is 24-inch. Normally, two blowers are adequate to meet aeration requirements in the Summer months. During late Fall and Winter as the solids concentration in the aeration basins is increased, three blowers are operated. When solids concentrations exceed 3,500 mg/l, the aeration system has been unable to maintain dissolved oxygen levels in the first two aeration basins. Until recent modifications discussed below, the fourth blower could not be operated because the discharge pressure would exceed the operating limits of the tube diffusers. The existing control system is primarily manual; individual blowers are started depending on oxygen demands as determined by the operator. Blowers are stopped by electronic timers when aeration requirement are minimal based on operating experience.

Deficiencies and Alternative Improvements

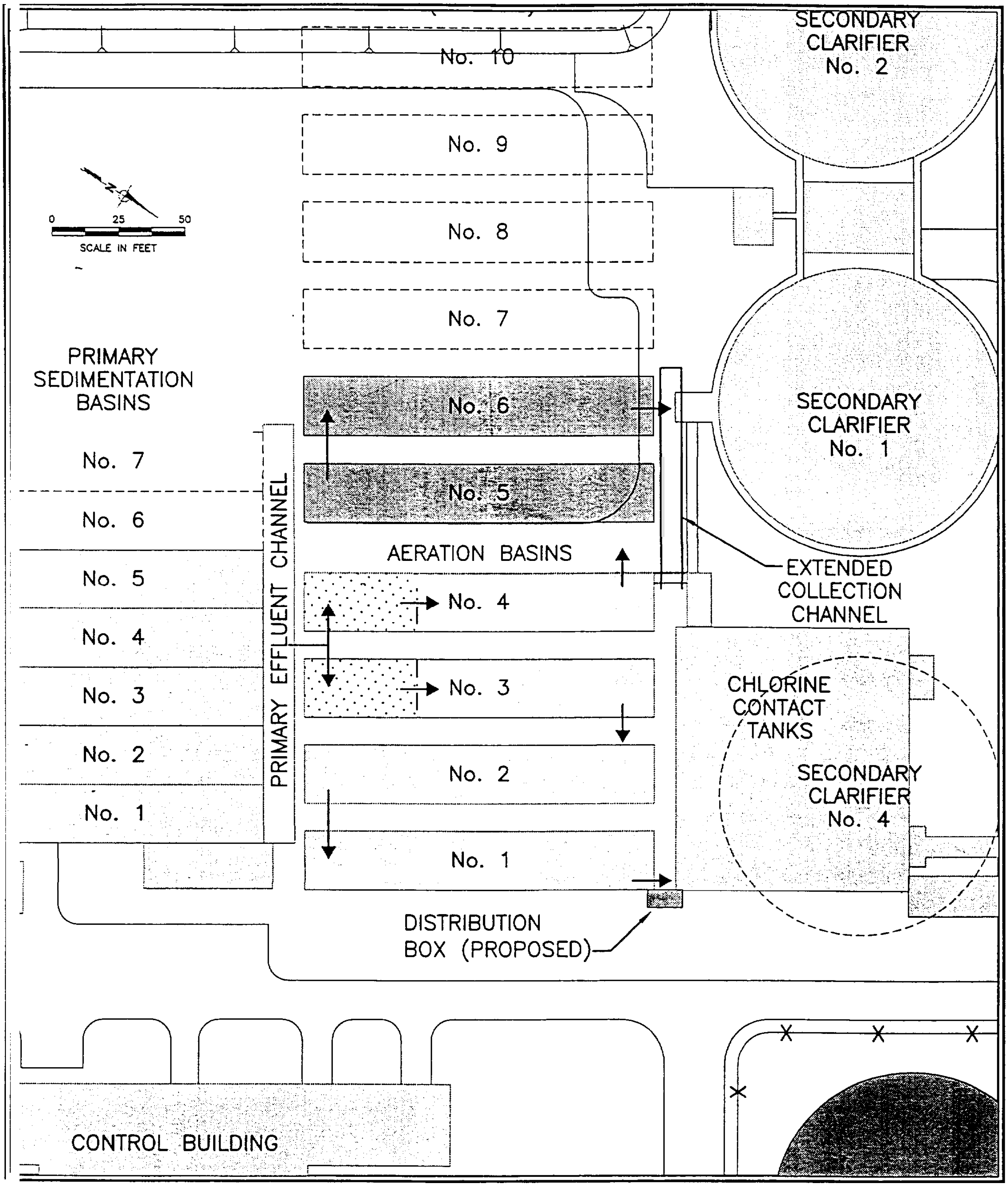
The secondary system was not designed for nitrification and needs several improvements to restore treatment capacity while providing for nitrification.

Aeration Basins. As of Fall 1999 when the aeration system was initially evaluated for this Master Plan study, air diffusion capacity in the aeration basins had been inadequate to maintain nitrifying conditions during Winter months. While air diffusion improvements may allow reliable operation with nitrification during the next few years, capacity will soon become limited by the minimum hydraulic detention time criteria during peak hour conditions and the limits of the secondary system to sustain an inventory of biomass necessary to provide nitrification.

As of Fall 1999, the existing aeration system was in need of immediate upgrade. The aeration system had proven to be inadequate in the Winter months with average dry weather flows in the range of 6.1 to 6.3 Mgd. The existing diffusers are evenly distributed between four basins. As a rule, the first half of the aeration basins in a series configuration requires 60 to 65 percent of the total oxygen demand. Sometimes this can be accomplished by adjusting the air flow to each basin or each header within the basin. This is possible where sufficient diffusers are in place to accommodate the increased air flow in a particular basin. Pushing excess air through an individual diffuser increases head loss and decreases oxygen transfer efficiency. Based on the initial review of the aeration requirements of the existing operation as part of the Master Plan study, it appeared that the diffuser system at Lodi was inadequate to provide the required distribution of air. The major deficiency was related to the diffuser system and not blower capacity, which is adequate for most existing operating conditions. The simplest solution was to replace the existing 24-inch diffusers with 36-inch diffusers in Aeration Basins No. 2, 3 and 4. A detailed description of the recommended changing of diffusers was contained in a summary report entitled Interim Aeration System Improvements dated October 21, 1999. A copy of this report is contained in Appendix [redacted]. This recommendation was implemented in December 1999 and has proven to be effective at current flow rates.

Additional aeration basins are required to maintain nitrification through the Master Plan planning period. The recommended upgrade is to construct two additional basins to provide capacity for full nitrification at 8.5 Mgd at an MLSS of 3,500 mg/L. The basins could be operated either as two parallel processes with three basins (Figure 11-5) each or as three parallel processes with two basins each (Figure 11-6). The beginning portion of one basin in each parallel process would be baffled off to serve as an anaerobic selector.

Primary effluent and RAS is currently discharged into the aeration basins in a zone which lacks any significant agitation or mixing. The aeration is turned off in this area to provide an anaerobic selector. As an interim measure, the aeration system could be operated at low output to provide a minimal level of mixing without adding significant oxygen. Mechanical mixing should be installed in the anaerobic selector zones when additional aeration basins are constructed.



Legend:

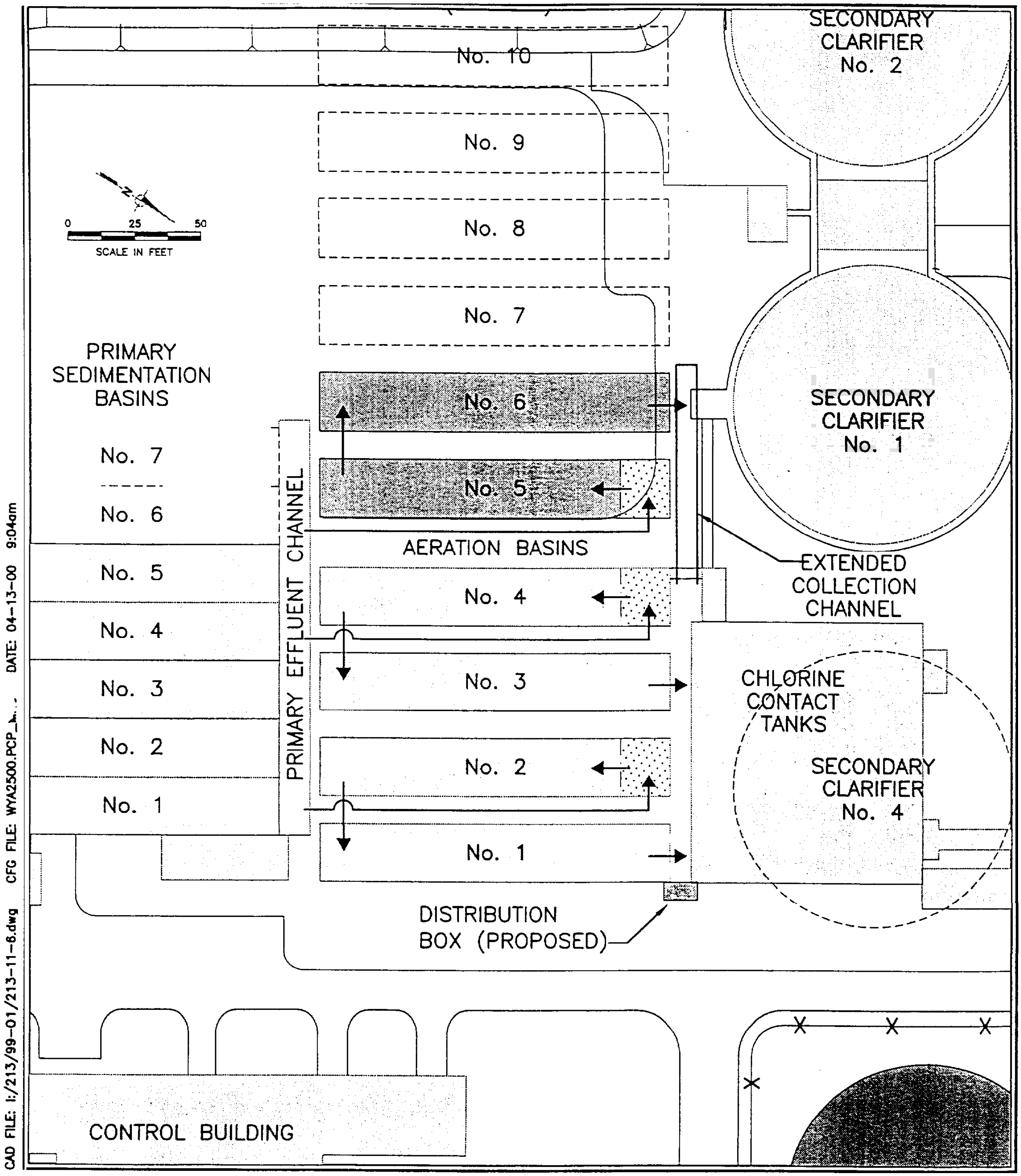
- Existing Facility
- Master Plan Facility
- Master Plan Plus Denitrification
- Anaerobic Zone
- Long–term Facility
- Baffle
- Flow Direction

Figure 11-5

City of Lodi
Water Pollution Control Facility

AERATION BASINS
TWO PARALLEL PROCESSES MODE

WEST YOST & ASSOCIATES



Legend:
Existing Facility
Master Plan Facility
Master Plan Plus Denitrification
Anaerobic Zone
Long-term Facility
Baffle
Flow Direction

Figure 11-6

City of Lodi
Water Pollution Control Facility

AERATION BASINS
THREE PARALLEL PROCESSES MODE

WEST YOST & ASSOCIATES

In conjunction with the construction of additional aeration basins, new primary effluent conveyance piping and channel modification will be required.

Air Supply. Air supply capacity is a critical component of secondary process capacity. At an ADWF of 8.5 Mgd, the maximum day air demand is estimated at about 10,000 scfm. The peak hour air demand is estimated at about 14,000 scfm. The existing firm air capacity is rated at 9,000 scfm with one blower out of service, so existing blower capacity is not adequate to meet projected Year 2020 aeration and nitrification needs.

The recommended project is to add two additional blowers at the same time new aeration basins are added. The air diffusion system in the new aeration basins should be consistent with blower capacity and oxygen requirements at Master Plan flow and loading conditions.

As the aeration basins and air supply are expanded, an automated control system using air manifold throttling valves controlled by dissolved oxygen concentration in the aeration basins would decrease future energy use by more closely matching the number of blowers operating to actual oxygen demands. The blowers would be controlled independently of the air manifold throttling valves, employing either variable speed drives or inlet throttling valves to maintain a preset outlet manifold pressure. Operating costs can and should be reduced through the use of increased blower control and higher efficiency blowers.

Secondary Clarifiers and Storage Ponds. With both clarifiers online, design peak hour surface loading rates of about 1,000 gallons per day per square foot at 8.5 Mgd are within the unit process design criteria previously presented in Table 11-1 (1,100 gpd/ft^2 max). The secondary clarifiers, therefore, have a capacity of at least 8.5 Mgd based on overflow rate. The capacity rating may be limited if the secondary clarifiers cannot accommodate the flow and solids loading delivered from the aeration basins. The entire secondary process (aeration basins, secondary clarifiers, and return activated sludge pumping) must therefore be evaluated as a whole.

The current secondary clarifiers lack redundancy. Although peak hour surface loading rates of about 1,000 gallons per day per square foot at 8.5 Mgd are within unit process design criteria, peak loadings will exceed 2,000 gallons per day per square foot, and solids loading would substantially exceed the system capacity with one unit out of service. Effluent quality at this loading will deteriorate to a significant degree, and sustained operation with one clarifier would likely "wash out" the secondary active cell mass. Alternate solutions are to construct a third clarifier, provide additional emergency effluent storage, or provide chemically enhanced clarification during operation with one clarifier.

Addition of a third clarifier is the only viable option. The best apparent site for construction of a new clarifier is east of the existing chlorine contact tank as shown in Figure 11-4. This site is not accessible to the existing clarifier influent distribution box, and the influent distribution box is designed for only two clarifiers. Therefore a new distribution box and new clarifier inlet piping are required for this alternative. Relocation of the chlorine contact tank would be required to accommodate construction of a fourth secondary clarifier as shown in the figure. The cost of a new clarifier and related improvements is estimated to be about $3,000,000.

Emergency effluent storage can be provided using the existing storage ponds. Clarifier effluent can be directed into a bypass pipeline and then conveyed into the ponds. The effluent can be returned through the equalization pond, irrigation pump station, and effluent return pipeline for treatment and disinfection after the emergency clarifier repairs had been performed. This could only be accomplished in the Winter months when the storage ponds were not being used for irrigation storage. It would be difficult to use the ponds in this manner without advance planning. There would be little or no direct cost associated with this alternative.

Chemically enhanced clarification could be provided with rehabilitation of the exiting polymer feed facilities and additional chemical feed facilities for metal salts. The capital cost of these facilities is estimated to be in the range of $100,000 to $200,000.

A chemical feed system for enhanced clarification should be installed as soon as possible as a short term means to provide emergency capacity. An operations plan should also be developed for the use of emergency storage should one clarifier need to be taken out of service for repair or maintenance. A third clarifier is really the only viable long-term option. The third clarifier should be constructed in conjunction with the construction of additional aeration basins.

RAS Pumping. The three RAS pumps can return a total of 8.6 Mgd, which is more than adequate for the Master Plan planning period. The most significant operating problems with the RAS pumping system are the lack of turn down capability and the inability to measure the rate of RAS withdrawal from each individual clarifier. The inlets to the RAS pumps are on a common manifold from the clarifiers with valves which allows either one pump to be used for both clarifiers or a separate pump to be used for each clarifier. The RAS meter is located downstream of the RAS pumps on the common RAS line discharging to the aeration basins.

In the past, treatment plant staff have tried to use one RAS pump for both clarifiers because one pump operating at normal speed provides the correct RAS flow rate. Unfortunately, the RAS header for one individual clarifier would tend to become partially clogged, and the RAS withdrawal from the other clarifier would then gradually increase. This condition allowed solids to accumulate in one clarifier. The current operating practice is to run a dedicated pump at low speed for each clarifier. Although this has provided satisfactory performance, the RAS pump curves should be evaluated to determine if trimming the impellers would provide better energy efficiency for this mode of operation.

Installing a flow meter on the RAS line from each clarifier would allow the plant operations staff to obtain better data for control of RAS return rates and the operation of each clarifier. New individual flow meters should be installed on each clarifier RAS pipeline in conjunction with the next major improvements to the aeration basins and clarifiers.

WAS Pumping. Waste activated sludge is pumped to the DAFT thickener each day for a period of about 10 hours. Thickened sludge is pumped to the digesters during thickener operation. Thickener performance could be improved by continuous operation 24 hours per day. Generally, biological systems are more stable when operated at a continuous solids withdrawal, and the DAFT thickener is a mechanical process which functions best if the feed and air dissolution systems are run continuously. Maximum thickener performance will become more critical as the volume of waste activated sludge increases in future years and the digester detention times are affected by the increased volume of waste solids.

Each WAS pump is rated at 160 gpm, which is excessive for continuous WAS withdrawal. Even at lower speeds, the pump performance is not suitable for continuous WAS withdrawal. WAS pumps should be downsized to allow continuous waste activated sludge withdrawal. Two of the four pumps should have their impellers trimmed or should be replaced with smaller units.

Summary of Recommended Facilities

Recommendations for new facilities associated with secondary treatment are summarized below:

- Replace the existing diffusers in Aeration Basins No. 2, 3, and 4 with 36-inch diffusers as soon as possible, (completed 12/99).
- Install flow meters on the RAS pipelines from each clarifier.
- Downsize two WAS pumps to allow continuous waste activated sludge withdrawal.
- Expand the aeration basins as required for projected increased loads and reliable nitrification. Two new basins are recommended.
- Provide a baffled anaerobic selector zone in the aeration basins. Remove existing aeration diffusers in this portion of the tank and install a slow speed mixing system to keep the solids in suspension.
- Add an automated blower control system using air manifold throttling valves controlled by dissolved oxygen concentration in the individual aeration basins.
- Construct a third clarifier as soon as possible.

Advanced treatment processes such as nutrient removal may require additional hydraulic improvements and more treatment tankage than the two additional aeration basins recommended above. The additional facilities for advanced treatment alternatives are discussed later in this section.

EFFLUENT DISINFECTION

The disinfection system uses chlorine to inactivate potential disease causing organisms remaining in the treated wastewater. The chlorine residual is converted to chloride ions through the addition of sulfur dioxide before the effluent is discharged to White Slough. The design criteria for chlorine disinfection were shown previously in Table 11-1. Effluent disinfection limits were discussed in Section 4.

Description of Existing Facilities

The disinfection system consists of a chlorination system, chlorine contact chamber, and a dechlorination system. The chlorination system includes chlorine gas storage, liquid and gas piping, chlorinators, injectors, and diffusers. The dechlorination system includes sulfur dioxide gas storage, liquid and gas piping, sulfonators, injectors, and diffusers. Chlorine and sulfur dioxide are stored outside in 2,700-gallon steel vessels.

The five chlorine contact basins are converted secondary sedimentation basins partitioned with redwood baffle walls to create a serpentine contact chamber. Basins are 80 feet long, 18 feet wide, and 8.2 feet deep with two baffle walls placed the length of each basin. Each basin volume is about 88,000 gallons. In a test performed in October 1999, average modal contact time with all basins in service was determined to be 93 minutes at 7.5 Mgd. This would translate to 82 minutes at the design flow of 8.5 Mgd. Peak hour contact time at Master Plan design flow of 16.3 Mgd is slightly over 40 minutes, which is adequate for secondary disinfection.

Assuming equalization storage to reduce diurnal peak flows, a tertiary level of disinfection would require 90 minutes modal contact time at peak day dry weather design flow. Modal contact time at the projected peak day dry weather flow of 10.0 Mgd is 70 minutes, which would not be adequate for tertiary treatment. An additional 130,000 gallons of tankage would be required to meet tertiary disinfection criteria.

There are two 2,000 lbs/day chlorinators for effluent disinfection and one 500 lbs/day chlorinator for filament, algae, and foam control. Total chlorine feed has ranged from 500 to 800 lbs/day during discharge to Dredger Cut. Chlorine feed has been less than 200 lbs/day during discharge to land. Chlorine residuals are normally maintained at between 3 and 4 mg/l. The sulfur dioxide system includes two 1,000 lbs/day sulfonators.

The existing disinfection capacity is based on a firm chlorinator feed capacity of 2,000 lbs/day, assuming one 2,000 lbs/day chlorinator is out of service. Dosages of up to 10 mg/L of chlorine are recommended for disinfection of activated sludge effluents. With a firm chlorination capacity of 2,000 lbs/day and a maximum dosage of 10 mg/L, the peak disinfection capacity is 24 Mgd. The system's firm capacity is, therefore, adequate to accommodate projected peak flow conditions for the Master Plan planning period. At a theoretical dose of 0.9 parts of sulfur dioxide per part of chlorine, the firm dechlorination capacity of 1,000 lbs/day is also adequate to accommodate the projected flow conditions.

Deficiencies

Chlorine doses in the 15 to 20 mg/L range may be required as a result of ammonia breakthrough due to incomplete nitrification in the secondary process. If secondary treatment capacity improvements are not implemented, it will be increasingly difficult to maintain adequate chlorination capacity. When ammonia breakthrough occurs, the initial chlorine demand (while the chlorination system attempts to maintain a free chlorine residual) could approach the chlorination system capacity or exceed it if one of the two chlorinators is out of service.

The existing chlorine contact basins provide adequate capacity at up to 11 Mgd ADWF for secondary disinfection. The basins would be inadequate to provide tertiary disinfection during the master planning period which requires 90 minutes modal contact time at peak day dry weather flow.

The disinfection performance of the current chlorination system has generally been satisfactory. Recent research indicates that disinfection performance and reliability can be improved by keeping chlorine contact tanks cleaned more frequently and by using microstrainers to remove any remaining large particles or flocs in secondary effluent prior to chlorination. Both of these measures serve to minimize solids in the chlorine contact tank which can shield bacteria from the chlorine disinfectant.

Several legislative or code requirements affect the manner in which chlorine and sulfur dioxide gas are stored and handled. This most significant code requirement is contained in the Uniform Fire Code (UFC). This is a model code produced by the International Conference of Building Officials. The UFC has no force of effect unless it has been adopted by a local agency. Section 80 of the UFC governs toxic gases stored under pressure, and applies to both chlorine and sulfur dioxide gas. Although the UFC has not been adopted by the local agency, compliance may be necessary to protect public safety and major revisions would be required to comply with the UFC, if chlorine gas and sulfur dioxide gas continue to be used.

General Chlorination Improvements

If chlorination continues to be the recommended method of wastewater disinfection, there are several improvements common to all the chlorination alternatives. These are as follows:

- Install valves, gates, and mixing enhancements to allow easier, more frequent flushing and cleaning of chlorine contact tanks.
- Expand contact tank volume by 130,000 gallons to provide 90 minutes modal contact time at PDWF for tertiary treatment (unless effluent discharge/disposal alternative without tertiary treatment is implemented). This assumes that equalization storage will be provided to eliminate hourly dry weather flow peaks.

The costs of these improvements will be added to the estimated costs of chlorination alternatives where appropriate.

Alternative Improvements

Given the restrictions imposed on the continued use chlorine gas and sulfur dioxide gas, a preliminary review was conducted of alternative means of providing effluent disinfection. The four alternatives that were evaluated included:

1. Improvements to the gaseous chemical based systems (chlorine & sulfur dioxide)
2. Liquid Chemical based systems (sodium hypochlorite & sodium bisulfite)
3. On-site generation of liquid chemicals
4. UV Disinfection

Chlorine Gas/Sulfur Dioxide Facilities Improvements. The treatment plant is currently equipped with a chlorine gas/sulfur dioxide disinfection system. This alternative would improve the existing system and include modifications and upgrades to enhance safety and reliability.

In order to comply with the Uniform Fire Code, the storage vessels must be enclosed and the new building must contain a treatment system to treat (scrub) air exhausted from the building in the event of the rupture of either a chlorine or sulfur dioxide cylinder. The treatment system must be sized to accommodate the largest storage vessel. In this case, the treatment system would have to accommodate 2,700 gallons of chlorine. It is assumed that a gas leak occurs at the point of connection at the top of the tank and that a catastrophic failure of the vessel does not occur. Alternatively, one ton cylinders could be used in place of one large vessel. Assuming that adequate space should be available for 30 days of chlorine and sulfur dioxide at average flow conditions and average dose, the required storage capacity would be as much as 10 1-ton cylinders. This would reduce the required scrubber capacity to a manageable size, but would require more space and more frequent handling of gas cylinders.

Sodium Hypochlorite/Sodium Bisulfite. In this alternative, the existing chlorination/ dechlorination equipment would be taken out of service and replaced with sodium hypochlorite and sodium bisulfite feed equipment. The sodium hypochlorite would provide disinfection in the contact tank while sodium bisulfite would be injected after disinfection to remove any toxic chlorine residual. Suppliers located in Stockton and Tracy produce sodium hypochlorite from chlorine gas and caustic.

The sodium hypochlorite system would be sized to provide the same firm capacity as the chlorine gas system – 2,000 lbs/day. Municipal-grade sodium hypochlorite is typically purchased as a 12½ percent solution. At this strength, there is 1 pound of chlorine in a gallon of solution. Approximately 2,000 gpd of sodium hypochlorite solution would be needed at peak design conditions.

Sodium hypochlorite decomposes over time. A 12½ percent solution loses half its strength in about 45 days. Therefore, in sizing hypochlorite storage facilities, a balance between maintaining an adequate reserve for peak demands and emergencies and minimizing the loss of chemicals through decomposition must be reached. Two new 10,000-gallon sodium hypochlorite storage tanks would be installed in the existing bulk storage area. The existing chlorine gas storage area could be split in half to provide enough space for each of the new tanks. At average design conditions, these tanks would contain 30 days worth of hypochlorite.

Sodium bisulfite functions as an equivalent to sulfur dioxide gas dissolved in water. The principles of dechlorination with sodium bisulfite are identical to those of sulfur dioxide. One pound of chlorine residual is removed by approximately 0.64 gallons of a 25 percent solution of sodium bisulfite. The feed system would be sized to eliminate 5 mg/L of chlorine at the peak flow of 16.3 Mgd – a bisulfite solution feed rate of approximately 500 gallons per day (gpd). Unlike sodium hypochlorite, sodium bisulfite solution can be stored for extended periods without significant chemical degradation. The storage tanks only need to be large enough to accommodate potential interruptions in delivery schedules. Two new 3,500-gallon sodium bisulfite storage tanks would be installed the existing bulk storage area. The tanks would have heat tape and insulation to prevent chemical precipitation. At average design conditions, the tanks would hold enough bisulfite solution for 30 days of operation.

All chemical storage tanks would be equipped with secondary containment to prevent spills. Metering pumps or eductors would withdraw hypochlorite and bisulfite solution from the storage tanks and inject it into carrier water. The carrier water streams would convey the hypochlorite and bisulfite to injection diffusers at the upstream and downstream ends of the chlorine contact basin, respectively. The control system would be similar to that of the other chlorination/ dechlorination alternatives except that the analyzers would control metering pumps or motorized valves rather than chlorinators and sulfonators.

Operating tasks would be similar to those of the chlorine gas. The primary difference would be the increase in chemical handling. On a weight basis, approximately 10 times more disinfectant would be needed. However, the sodium hypochlorite would be delivered in tank trucks and pumped directly to the storage tanks. Use of a sulfur dioxide storage vessel and maintenance of sulfonators would be replaced with filling sodium bisulfite storage tanks and maintaining metering pumps or eductors.

Because chlorine gas and sulfur dioxide would be replaced with sodium hypochlorite and sodium bisulfite solutions, the use of hazardous gases would be eliminated. This offers significant safety advantages. Secondary containment would prevent spills from creating a safety hazard.

On-Site Generation. In this alternative, sodium hypochlorite is produced on site from salt, water, and electricity. Softened water and salt are mixed to produce a concentrated brine solution. The solution is then diluted and passed through an electrolytic cell where electrolysis converts the sodium chloride to sodium hypochlorite. The final product contains approximately 8,000 mg/l of sodium hypochlorite (0.8% solution). Byproducts include hydrogen gas and unreacted salt at a concentration reported by one manufacturer to be 20,000 mg/l. This is slightly greater than purchased liquid sodium hypochlorite. Hydrogen gas is constantly vented to avoid buildup. On-site generation facilities have the advantage of producing a relatively stable product on as-needed basis, and it eliminates the hauling and transfer of liquid chlorine.

On-site generation facilities are becoming popular for small water facilities in the range of 100 pounds per day. Larger generating facilities up to a few thousand pounds per day of chlorine are more common in the southeast and in southern California. The City of Folsom recently constructed and started operation of a 600 lbs/day on-site generation facility at the Folsom Water Treatment Plant. Preliminary results have been favorable. The facility is too new to have any experience with actual operating costs.

In other aspects this on-site generation of sodium hypochlorite is similar to the sodium hypochlorite/sodium bisulfite alternative in such aspects as chemical storage, chemical feed, and dechlorination with sodium bisulfite The sodium hypochlorite generating system could be sized to provide the average chlorine demand with peak demands provided from storage. Proposed generating capacity is 1,200 lbs/day in two or more units. Peak feed capacity would be the same firm capacity as the chlorine gas system – 2,000 lbs/day. Sodium hypochlorite could be purchased as a backup supply of chlorine.

The primary difference to the liquid sodium hypochlorite alternative would be the decrease in chemical handling. On a weight basis, approximately 3.5 pounds of salt is required to produce one pound of chlorine as compared to 8.4 pounds of liquid sodium hypochlorite to provide one pound of chlorine. Dry salt is delivered in tank trucks and blown into the brine tanks. A new building would be required with room for the brine tanks, bisulfite tanks, and three generators on 6-foot by 12-foot pads.

UV Disinfection. UV light with a wavelength of 254 nanometers penetrates the cell walls of microorganisms and prevents reproduction by altering the structure of the microorganism's DNA. UV is well suited to activated sludge treatment plants with relatively long mean cell residence times (MCRT) such as are used at Lodi. The long MCRT breaks down particle associated coliform bacteria, which are the most difficult ones to reach with UV light. A delivered UV dosage of approximately 100 mJoules/cm^2 at peak day flow is needed for either secondary (23 MPN/100 mL) or filtered tertiary (2.2 MPN/100 mL) disinfection.

Conventional UV disinfection systems use low-pressure, low-intensity mercury vapor lamps, which are similar to the fluorescent lamps commonly used for commercial lighting. While low-pressure lamps are efficient at producing germicidal light, they are rated for relatively low power use – about 40 watts. Therefore, relatively large numbers of lamps are required to disinfect a given effluent flow rate—on the order of 50 lamps per Mgd of peak flow depending on effluent quality and disinfection requirements.

With the advent of medium-pressure, high-intensity lamps, UV disinfection is becoming a more attractive alternative at large WWTPs. While medium-pressure lamps are less efficient than their low-pressure counterparts, their higher power use (approximately 2 kilowatts), greatly reduces the total number of lamps needed to treat a given flow rate. Medium-pressure systems typically have one-tenth to one-fifteenth as many lamps as low-pressure systems of the same capacity. Modern medium-pressure systems are normally equipped with automatic lamp cleaning devices, further reducing maintenance requirements. However, medium-pressure systems do have several drawbacks. In addition to lower efficiency, medium-pressure lamps cost more and last less than half as long as low-pressure lamps. Despite these disadvantages, most new UV disinfection systems designed and constructed at medium and large plants are medium-pressure due to the reduced maintenance requirements and smaller system footprint.

A third UV disinfection option is the recently developed low-pressure, high-intensity lamp. These lamps have the high efficiency and long lamp life associated with low-pressure, low-intensity lamps; however, their higher output results in system designs that require only one-third as many lamps as conventional low-pressure, low-intensity systems. In addition, much like medium-pressure systems, low-pressure, high-intensity systems are equipped with automatic lamp sleeve cleaning systems to reduce labor costs.

A new UV disinfection system would consist of the following components:

- Channel structure. At least two channels are needed so that half the system can be taken out of service for maintenance. The channels could be located indoors or outdoors, or constructed within the existing chlorine contact basin.
- Control system. The control system can be located in panels rated for outdoor installation or be installed indoors to simplify maintenance during inclement weather.

- Lamp modules. Several lamps would be located on a single module. Each module can be removed from the channel independently. Lamp ballasts are located directly above the modules.

- Automatic lamp cleaning mechanism. The efficiency of the UV system deteriorates as deposits build up on the lamp sleeves. Automatic wiper mechanisms typically use a chemical solution to help remove deposits on the lamp sleeves.

- Level control gates. A level control gate would be installed in the downstream end of each channel. The gates control water level and velocity. It should be noted that medium-pressure UV systems create over 2 feet of head loss, so a detailed plant hydraulic evaluation is critical.

- Sodium hypochlorite system. A small liquid hypochlorite feed system would be needed for other miscellaneous chlorine needs at the plant.

Operation and Maintenance associated with UV disinfection systems include:

- Lamp replacement. Medium-pressure lamps have an expected life of 5,000 hours. However, as with other lights, the useful life of UV lamps is also affected by the number of on/off cycles. Lamp ballasts also must be replaced periodically; however, this cost is small relative to that of lamp replacement.

- Maintenance of the automatic wiper mechanism. The cleaning solution reservoir must be refilled regularly. The lamp sleeve wiper mechanism must be periodically realigned to prevent binding.

- Periodic manual lamp sleeve cleaning. The lamp sleeves may have to be cleaned manually on occasion. The need for manual cleaning is related to the performance of the WWTP. For example, manual cleaning probably would be necessary after a major plant upset.

Screening of Disinfection Alternatives

Disinfection alternatives are compared on relative cost differences and on a non-economic evaluation.

Chlorine Gas/Sulfur Dioxide Improvements Costs. Construction of new gaseous chlorine facilities would be less costly than UV disinfection but more costly than the use of liquid sodium hypochlorite and sodium bisulfite. As the area around the treatment plant develops, the need to contain an accidental release of chlorine will become more critical. Already, risk associated with exposure to traffic along I-5 is significant. The cost of conversion to 1-ton cylinders and containment and treatment systems is estimated at approximately $1.4 million. Additional chlorine contact tank capacity for tertiary treatment would add another $1.1 million for a total of $2.5 million.

Sodium Hypochlorite/Sodium Bisulfite Facilities Costs. The costs of sodium hypochlorite/ sodium bisulfite feed facilities and additional chlorine contact tanks are estimated at approximately $1.3 million, which is less than comparable gas containment facilities or UV disinfection. Purchase of liquid chlorine or on-site generation costs are similar over the long term. Liquid chlorine feed facilities are less expensive to construct, but liquid chlorine costs more to purchase. On-site generation facilities are more expensive to construct, but the cost of chlorine production is less than liquid chlorine hauled to the treatment plant site. On-site generation appears very attractive in areas remote from sodium hypochlorite supply outlets or on sites where bulk storage of liquid sodium hypochlorite is undesirable. These conditions do not currently exist in the Lodi area where liquid sodium hypochlorite is produced both in Stockton and Tracy.

The addition of sodium hypochlorite to effluent tends to increase pH slightly while the addition of chlorine gas tends to decrease pH slightly. The treatment plant has had a few occasions of effluent pH below the minimum specified in the discharge permit (6.5). The use of sodium hypochlorite would have a beneficial effect in ameliorating low effluent pH when compared to the continued use of chlorine gas for disinfection.

On-Site Liquid Chemical Generation Costs. The estimated cost of producing one pound of chlorine from 3.5 pounds of salt and 2.5 kW hours of electricity per pound of chlorine is about 42 cents based on salt at 4 cents per pound and electricity at 11 cents per kWh. This costs does not account for electrolytic cell replacement or maintenance which one supplier estimated at 3 cents per pound of chlorine produced. The facilities required for delivered liquid sodium hypochlorite and on-site generated sodium hypochlorite are similar regarding storage tanks, feed pumps, and dechlorination. On-site generation in addition to these facilities requires two generators about 6 by 12 feet in dimension and two 10-foot diameter brine tanks. The additional cost of these facilities, above and beyond the cost of liquid sodium hypochlorite facilities common to both alternatives, is estimated at about $700,000 including additional building space. This equates to about 20 cents additional cost per pound of chlorine produced over 15 years at a 6 percent discount rate assuming chlorine demand at about 850 pounds per day and continuous over the life of the generator. Therefore, for a relative comparison of costs the on-site generation of chlorine costs about 67 cents per pound of chlorine. This includes the cost of materials, energy, and additional generating facilities not required for liquid sodium hypochlorite or sodium bisulfite dechlorination. This is not a total cost but the material cost and incremental cost increase for on-site generation facilities.

The material cost of one gallon of liquid sodium hypochlorite which provides about one pound of chlorine is 50 to 75 cents per gallon delivered in bulk. Chemical suppliers would not provide a firm estimate. The actual cost will depend on the quantities ordered, market conditions at the time of bidding, and other conditions of the purchase. Overall, on-site generation of chlorine may cost about the same as liquid sodium hypochlorite. Based a preliminary cost review of alternatives, on-site generation of chlorine compares favorably to liquid sodium hypochlorite over the life of the project. The differences is that on-site generation of chlorine involves unknown operation and maintenance costs and reliability issues whereas liquid sodium hypochlorite involves unknown cost increases that may be passed on to the City from the supplier as a result of future market conditions.

UV Irradiation System Costs. Based on other studies, the capital cost of UV disinfection facilities is two to three times the cost of other alternatives. This is especially true where the existing facility already has a chlorine contact chamber associated with other disinfection alternatives. This cost is somewhat offset because there is no need to buy chlorine or other products. This advantage is somewhat less significant at Lodi where disinfection is not provided for land application of effluent during the Summer. Filters may be needed for the UV system to comply with even a 23 MPN/100 mL disinfection limit. The estimated cost for UV disinfection for Lodi is over $4,000,000. Overall, the cost of UV disinfection makes this alternative uneconomical at this time. UV disinfection systems are still evolving and may be a desirable disinfection in the future, especially if complete effluent filtration is installed, and the existing chlorine contact tanks need to be replaced with new, larger capacity tanks.

Costs and Subjective Evaluation Criteria Comparison. Selection of a disinfection alternative will depend to a great extent on non-economic considerations. The costs and relative rankings of subjective evaluation criteria are shown in Table 11-5. The estimated costs shown in Table 11-5 are preliminary and for comparison purposes only. The costs include engineering, administration, contingency, and escalation to ENR 7000. Rationale for the rankings are summarized in Table 11-6. The non-economic evaluation factors were presented and explained in Section 6.

Table 11-5. Costs and Subjective Comparison of Disinfection Alternatives

		Alternatives			
Subjective Criteria	Weightings	Gas Chlorination	Liquid Chlorination	On-Site Generation	UV
Compliance with Future Discharge Requirements	1.5	3	3	3	4
Reliability	1.5	4	3	2	3
Flexibility	1.5	3	4	3	3
Ease of O&M	1	4	5	3	3
Ease of Implementation	1	3	4	3	2
Environmental Impacts	1	1	2	2	4
Safety	1	1	3	4	4
Aesthetics	0.5	2	3	3	4
Totals		21	27	23	26
Totals (weighted)		24	29	24	28
Capital Cost (secondary)		$1,400,000	$260,000	$960,000	$4,800,000
Capital Cost (tertiary)		$2,500,000	$1,300,000	$2,000,000	$4,800,000
Annual O&M (tertiary)		$141,000	$305,000	$270,000	$275,000
Life Cycle Cost (tertiary)		$4,000,000	$4,500,000	$4,900,000	$7,700,000

Note: Present worth of O&M costs calculated at a discount rate of 7% for 20 years.

Table 11-6. Basis of Subjective Rankings for Disinfection Alternatives

Alternative	Safety	Reliability	Flexibility	Ease of Operation	Environmental Impacts	Implementation
1. Chlorine Gas/Sulfur Dioxide	• Hazardous gases used • Scrubber/containment system contains leaks in building • Safety concerns during transport and handling of cylinders	• Proven process • Residual allows confirm-ation of disinfection • Can comply with disinfection standards during process upsets by increasing doses • Ammonia and nitrites can significantly increase chlorine demand and impair operation	• Multiple units permits maintenance during low flows • Chlorine can be used for process and odor control • Can comply with reclaimed water disinfection standards	Identical to existing system	• Toxic chemicals are used • Potential for discharge of toxic residuals • Potential for formation of toxic/carcinogenic byproducts • Low energy use	Gas scrubbing system must be added, including conversion to 1-ton cylinders
2. Sodium Hypochlorite/ Sodium Bisulfite	• Hazardous liquids used • Spill containment required • Safety concerns during transport and handling of chemicals • Ventilation required to remove fumes	• Proven process • Residual allows confirm-ation of disinfection • Sodium hypochlorite decomposes over time • Can comply with disinfection standards during process upsets by increasing doses • Ammonia and nitrites can significantly increase chlorine demand and impair operation	• Multiple units permits maintenance during low flows • Chlorine can be used for process and odor control • Can comply with reclaimed water disinfection standards	Relatively simple control system	• Toxic chemicals are used • Potential for discharge of toxic residuals • Potential for formation of toxic/carcinogenic byproducts • Low energy use	• Existing equipment abandoned • Chlorine contact basin has large footprint • Multiple equipment manufacturers and chemical suppliers increases competition and lowers cost • No scrubbing/gas containment needed
3. Sodium Hypochlorite/ Sodium	• Hazardous liquids used • Spill containment required • Ventilation required to remove fumes	• Proven process but more complex operation • Residual allows confirmation of disinfection • Sodium hypochlorite does not decompose over time • Can comply with disinfection standards during process upsets by increasing doses • Ammonia and nitrites can significantly increase chlorine demand and impair operation	• Multiple units permits maintenance during low flows • Chlorine can be used for process and odor control • Can comply with reclaimed water disinfection standards	Relatively complex system	• Toxic chemicals are used • Potential for discharge of toxic residuals • Potential for formation of toxic/carcinogenic byproducts • Medium energy use	• Existing equipment abandoned • Chlorine contact basin has large footprint • Limited number of large systems in California • No scrubbing/gas containment needed

Table 11-6. Basis of Subjective Rankings for Disinfection Alternatives *cont'd...*

Alternative	Safety	Reliability	Flexibility	Ease of Operation	Environmental Impacts	Implementation
4. UV Irradiation	• No hazardous chemicals used • Exposed UV lights can impair vision	• Relatively new process • No measurable residual to confirm disinfection • High effluent solids impairs performance • Process unaffected by ammonia and nitrites	• Dual channels permit maintenance during low flows • Parallel channels can be added to increase capacity • Must have separate hypochlorite system for other chlorine needs • Major upgrade may be needed to comply with reclaimed water disinfection standards	• Relatively complex control system • Automatic lamp cleaning device may require significant maintenance • Operators unfamiliar with system • Effluent filtration system required	• No toxic chemicals used • High energy use • No toxic residuals • No toxic/carcino-genic byproducts formed • Effluent filtration system would reduce BOD and TSS discharges	• Existing chlorination system would be abandoned • Small system footprint • Only one manufacturer of large systems Lack of competition may increase costs • Existing chlorine building can be used for equipment and materials storage

Recommended Secondary Disinfection Facilities. For either tertiary or secondary treatment, the liquid sodium hypochlorite and sodium bisulfite based system is recommended for the following reasons:

1. The hypochlorite/bisulfite option had the highest subjective ranking and was close to the lowest life cycle cost.

2. Serious safety concerns are involved with chlorine gas use and handling.

3. A liquid sodium hypochlorite and sodium bisulfite system is less expensive and less labor intensive to maintain than UV disinfection and less expensive than construction of new gas chlorine handling facilities. The low capital costs required for a liquid chemical system would allow future conversion to UV or other disinfection technology without requiring the abandonment of expensive fixed facilities.

4. A liquid sodium hypochlorite and sodium bisulfite system is less complex than on-site generation of sodium hypochlorite, and there are less labor and maintenance costs associated with liquid sodium hypochlorite.

Sodium hypochlorite facilities should be constructed to replace the existing gaseous chlorine facilities. Sodium bisulfite facilities should be constructed to replace the existing gaseous sulfur dioxide facilities. The sodium hypochlorite will be used as a source of chlorine for effluent disinfection, clarifier weir maintenance, foam control, return sludge chlorination for bulking control, and for pre-chlorination of the influent wastewater at the influent junction box to control odors.

At least 30 days of on-site storage should be provided for each chemical. Tanks for the sodium bisulfite are anticipated to be of cross-linked high density polyethylene construction. Tanks for sodium hypochlorite should be high density linear polyethylene construction. All tanks should be located in the bulk storage area with secondary containment walls. Separate chemical metering pump feed systems should be provided for each chemical and housed in an adjacent building. New facilities should provide for easy future conversion to on-site generation of sodium hypochlorite or UV disinfection.

EQUALIZATION PONDS AND EFFLUENT PUMPING

Treated effluent is currently either discharged to Dredger Cut or is applied to City owned land for irrigation of animal feed and fodder crops. Treated effluent for irrigation flows through one or more ponds prior to application to land. The storage and equalization ponds are described in this section.

Equalization Pond

When not discharging to Dredger Cut, treated effluent is conveyed to the 3.5 million gallon equalization pond by gravity flow and then either pumped to the irrigation system by the irrigation pumps or pumped into the effluent storage ponds by the recirculation pumps until needed for irrigation. The irrigation pumps may also be used to return stored effluent directly to the wastewater treatment process. A schematic diagram of the pond system was included in Figure 11-2. The pond layout was shown previously in Figure 11-3.

The equalization pond may also be used to collect untreated wastewater for emergency storage by lowering the equalization pond water level and opening the bypass gate at the Water Pollution Control Facility headworks. Untreated wastewater will then flow by gravity to the equalization pond and may be pumped to the effluent storage ponds as required. The equalization pond serves as a large wet well for the irrigation pump system.

Irrigation and Recirculation Pump Stations

The irrigation pump station consists of three 3,500 gpm vertical turbine pumps and one smaller vertical turbine pump mounted on a concrete structure located over the equalization pond water surface.

The recirculation pump station consists of two vertical turbine pumps for transferring treated effluent from the equalization pond to the storage ponds. The recirculation pump station also includes a submersible pump for pumping irrigation tailwater to the storage ponds. Irrigation tailwater flows by gravity from the effluent irrigation area to the recirculation pump station. The tailwater is combined municipal and industrial effluent runoff from the irrigated fields.

Aerated Lagoon and Skimming Pond

Two of the storage ponds located adjacent to and west of the equalization pond were originally referred as the Aerated Pond and the Skimming Pond. These were originally used for industrial wastewater treatment prior to storage. When used for industrial wastewater treatment, the ponds became overloaded and filled with wastewater solids. These ponds were taken out of service several years ago. Industrial wastewater is now applied directly to land without pond treatment or storage.

Deficiencies and Recommended Improvements

Modification of the pond system is required to accommodate construction of a tertiary filtration system and eventually new chlorination tanks. At a minimum, this construction would require relocation of the equalization pond south berm. The costs of tertiary filtration will include the costs for relocating the equalization pond south berm.

Another area of needed improvement is associated with the tailwater return system. The tailwater is typically anaerobic by the time it flows to the pump station and as a result can cause odor problems in the storage ponds. The preferred method for tailwater return is to construct a new tailwater return pump station west of the Water Pollution Control Facility at the collection point for the effluent irrigation area runoff (see Figure 11-7). Collected runoff would then be returned to the storage ponds in a small force main. This would eliminate the gravity return pipeline and the need to pump tailwater at the recirculation pump station. This would help with odor problems associated with the tailwater. Further reductions in the volume of tailwater would also be helpful.

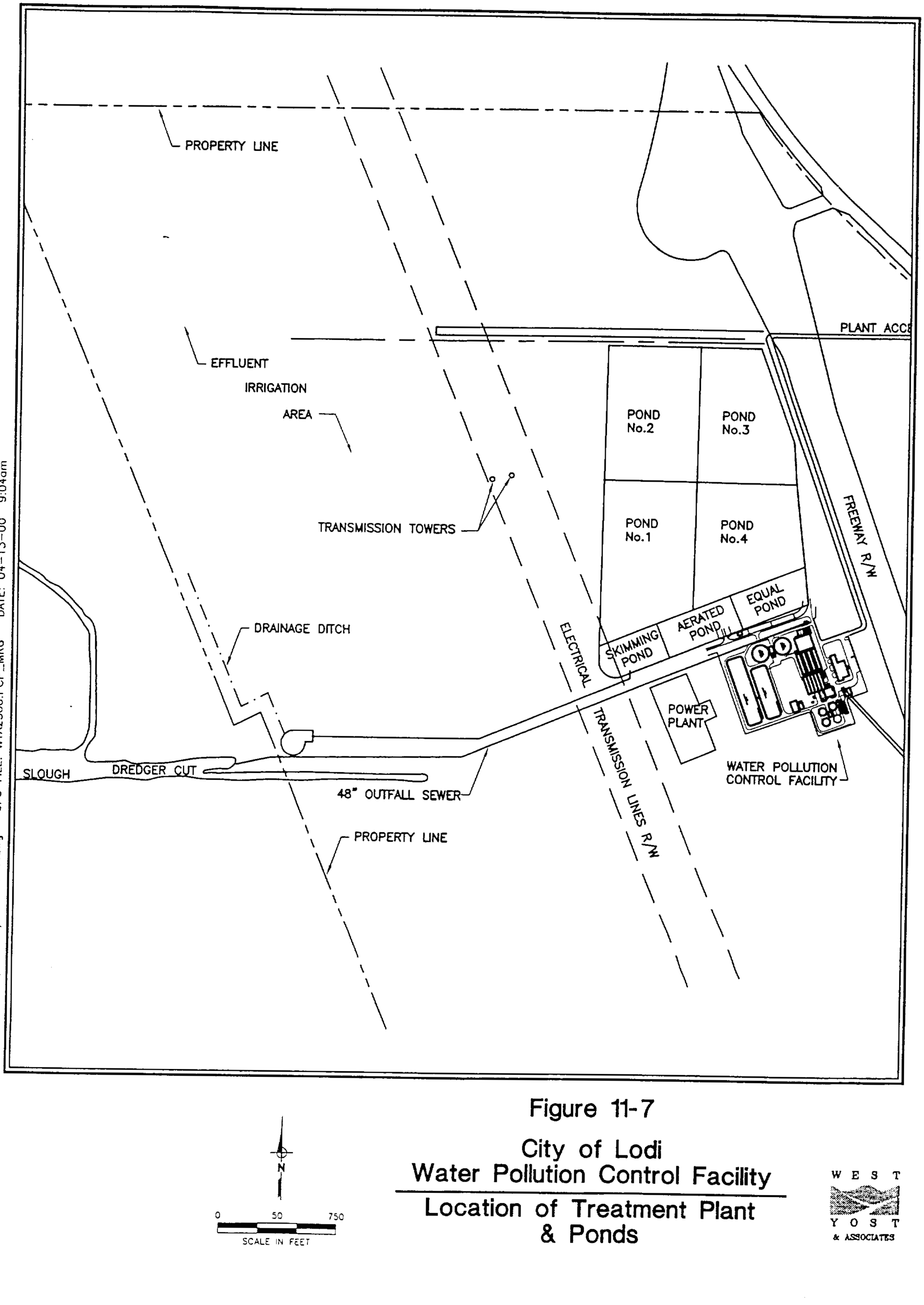


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Figure 11-7

City of Lodi
Water Pollution Control Facility

Location of Treatment Plant
& Ponds

WEST
YOST
& ASSOCIATES

STORAGE PONDS

Description

There are four storage ponds north of the treatment plant used for the storage of treated effluent (see Figure 11-2). These ponds are used during the growing season to equalize the effluent flows with irrigation demands. The ponds are used during the Winter to store tailwater runoff and to provide some effluent pH and temperature moderation.

Deficiencies and Recommended Improvements

The storage pond control piping constructed in 1990 is working properly and provides sufficiently flexibility for all current modes of operation. If the ponds were to be used for emergency effluent storage when one of the two existing secondary clarifiers was temporarily out of service, some valving improvements would be recommended to be able to more easily return water to the chlorine contact tank. The dissolved oxygen level in the ponds can drop below 1.0 mg/L when there is a relatively high amount of industrial effluent or tailwater from irrigation with industrial effluent entering the ponds. Floating brush style aerators are recommended for the storage ponds to maintain higher dissolved oxygen levels and help prevent odors from the ponds. Although it is difficult to predict the tailwater BOD and therefore the required aeration horsepower, approximately two 15 hp aerators per pond should generally provide sufficient aeration to keep the dissolved oxygen in the upper portion of the ponds above 1.0 mg/L.

SOLIDS THICKENING

Solids residuals removed from the liquid wastewater stream are treated by anaerobic digestion prior to disposal. Primary sludge is thickened in the primary sedimentation basins as previously described. Waste activated sludge (WAS) from the secondary process is thickened in a separate process facility to remove excess water before being pumped to the digesters.

Description of Existing Facilities

WAS is thickened in a rectangular dissolved air flotation thickener (DAFT) where particles attach to microscopic air bubbles and float to the surface for removal by skimming. Heavier particles settle to the bottom of the thickener tank. Sludge removed by flotation or settling is pumped to the digesters, and the remaining effluent (subnatant) is returned to the industrial influent flow in the plant headworks channel. No effluent is returned to the domestic treatment plant.

WAS thickening can be expected to increase the solids concentration to between 3 and 4 percent. No actual data are collected at Lodi. The solids recovery rate is typically in the range of 85 to 95 percent without the use of polymer. Polymer can increase the recovery rate, but the polymer feed facilities added in 1976 for sludge dewatering are no longer in service.

DAFT No. 1 was constructed in 1976 with a surface area of about 300 square feet. DAFT No. 2 was constructed in 1990 with a surface area of about 600 square feet. Under the current operation, WAS is thickened in DAFT No. 2 only for a period of about 10 hours per day. DAFT No. 1 is out of service. DAFT operation is intermittent because the WAS pumps are oversized and do not allow 24-hour per day pumping.

Typical DAFT loading rates for WAS are 10 to 30 lb/sf/d (if polymer is used). Based on 12 lb/sf/d, the capacity of DAFT No. 2 is about 7,200 lb/day. WAS solids production is projected to average 10,500 lb/day depending on influent BOD concentrations and actual solids yield. The existing loading on DAFT No. 2 probably exceeds the equivalent of 24 lb/sf/d for the 10 hour per day operation.

Solids Balance

An estimate of the solids production from the liquid stream process units and the subsequent loadings on the solids handling processes was made in order to determine the design loadings on the solids handling units. The estimates were made based upon typical (or actual, if available) process performance and solids characteristics assumptions as follows:

1. The primary sedimentation basins remove approximately 65 percent of the influent suspended solids from the main plant flow stream and thicken it to an average concentration of 5 percent prior to pumping to the digesters. This solids stream was assumed to contain a volatile fraction of 70 percent.

2. The primary sedimentation basins remove approximately 30 percent of the influent BOD from the main plant flow stream before discharge to the secondary system. Solids production estimates from the secondary treatment systems were made by scaling the output of a Biowin model for a nearly identical process under design for the City of Vacaville. The model assumed full nitrification, a mean cell residence time of 9.5 days and a MLSS concentration of 3500 mg/L.

3. The secondary solids were assumed to contain a volatile fraction of 80 percent. The concentration of this flow stream used in this analysis assumed that solids will be obtained from the return sludge piping from the secondary clarifiers at concentrations around 0.8 percent solids.

4. Solids capture in the dissolved air flotation thickeners (DAFTs) was assumed to be 95 percent of the solids they receive. Thickened solids concentration was projected to be 3.5 percent.

5. Primary and thickened waste activated sludges are combined and placed in the anaerobic digesters for stabilization and solids destruction. The digesters are expected to achieve a 50 percent rate of destruction of the volatile solids fraction.

6. Digested solids are then sent to the sludge storage lagoons. Volatile solids destruction in the storage lagoon was assumed to be 30 percent.

The resulting flow, total suspended solids, and volatile solids loadings on the main plant components at average annual and maximum month influent loading conditions for Year 2020 are shown in Table 11-7. The detailed solids balance is shown in Appendix [redacted].

Table 11-7. Projected Solids Production

Parameter	Average	Max Month
Plant Loadings		
BOD, lbs/day	19,800	22,800
TSS, lbs/day	17,600	22,400
Primary Solids		
Total Solids, lbs/day	11,300	14,400
VSS @ 70%, lbs/day	7,900	10,100
Total Flow @ 5%, gpd	27,200	34,600
Secondary Solids		
Total Solids, lbs/day	10,500	12,100
Thickened Solids, lbs/day	10,000	11,500
VSS @ 80%, lbs/day	8,600	9,300
Thickened Sludge Flow @ 3.5%, gpd	34,200	39,500
Digester Feed		
Total Solids, lbs/day	21,300	25,900
VSS, lbs/day	16,500	19,400
Total Flow, gpd	61,400	74,000
Digested Solids		
Total Solids, lbs/day	13,000	16,200
Total Solids, tons/year	2,370	—
Total Flow, @ 2.6%, gpd	61,400	74,000

It should be noted that the primary sludge flow as measured by the sludge flow meters does not match the primary sludge flow calculated from TSS removal and measured percent solids. Despite several tests of the sludge flow meters, the cause of this discrepancy has not been discovered. The result is that the projected solids production values used in this section are significantly higher than the measured and reported quantities of biosolids used for calculations in Section 9.

Deficiencies and Alternative Improvements

Alternative solids thickening deficiencies and alternative improvements are described below.

Continuous Operation. The most significant operating deficiency with the WAS thickeners is the inability to operate 24 hours per day. Operation should be changed to 24 hours per day after the capacities of the WAS pumps are reduced. Continuous operation will reduce the solids loading and should improve thickener performance.

Capacity. DAFT No. 2 for WAS thickening is currently operating at double the recommended loading rate for less than half of each day. It will be moderately overloaded at the projected master plan WAS solids production and has no operational redundancy. If the DAFT No. 2 is taken out of service, either a primary clarifier must be temporarily converted to WAS thickening or the digesters must accommodate a diluted sludge flow for several days. Available alternatives are to either rehabilitate DAFT No. 1 or construct a new thickener. The preferred alternative is to replace unit No. 1 with a new thickener equal in capacity to Unit No. 2. This alternative would provide two similar units and improve reliability.

Co-Thickening of Primary Sludge and WAS. Co-thickening of these sludges has been successfully practiced at other wastewater treatment plants. For co-thickening, primary sludge is removed from the primary clarifiers continuously at a relatively low solids concentration (approximately 1 percent). This primary sludge is combined with WAS before thickening in the DAFTs. The reported benefits are:

- A thicker sludge float
- Better primary BOD removal
- Better primary TSS removal at high flows

These benefits can translate into slightly lower capacity requirements for primary clarification, secondary aeration, and anaerobic digestion. Co-thickening does not require any increase in DAFT sizing, but does require more than doubling the pressurized recycle and air flows to maintain a high air:solids ratio.

A co-thickening trial was run at the City of Vacaville Easterly plant in 1998. The performance results were generally favorable, but operation was substantially more difficult and odorous than with WAS alone. The projected energy costs for co-thickening were approximately $70,000 per year greater than the costs for treating the expected 5 percent higher BOD loads to the secondary treatment system if co-thickening were not practiced. Co-thickening will require covers, foul air collection, and odor scrubber. Based on the trial run and projected operating costs, it was determined that co-thickening would not be cost-effective unless anaerobic digestion capacity or secondary aeration capacity became limiting. The elimination of thickening in the primary clarifiers may also reduce the concentrations of volatile fatty acids in primary effluent, which in turn could reduce the secondary solids settleability and phosphorus removal. In the case of Lodi, expansion of the anaerobic digestion capacity and secondary aeration capacity are required regardless of any benefits achieved by co-thickening. As a result, co-thickening is not recommended for Lodi.

Effluent Return. The DAFT effluent (subnatant) from the WAS thickener is returned to the industrial influent flow at the plant headworks channel. Irrigation of animal feed crops with DAFT subnatant may be allowed at the present, but it does increase the risk of pathogen transfer to farm workers and may not be allowed under future reclamation requirements. New piping should be installed to return all DAFT subnatant to the treatment plant headworks. Polymer addition may be provided to improve solids capture and reduce the amount of solids returned to the treatment system.

Summary of Recommended Facilities

Recommendations for new facilities associated with WAS thickening are summarized below:

- Operate the WAS thickener continuously for 24 hours per day.
- Construct a second DAFT similar in capacity to DAFT No. 2.
- Continue to thicken primary sludge in the primary clarifiers.
- Return all DAFT subnatant to the treatment plant headworks.
- Restore the DAFT polymer feed system.

SOLIDS STABILIZATION

Solids from the primary clarifiers and DAFTs are anaerobically stabilized to reduce pathogens and convert the solids to a stable product suitable for storage and direct land application.

Description of Existing Facilities

Primary sludge, thickened waste activated sludge, and scum is pumped alternately to Primary Digester Nos. 1, 2 and 3 on an hourly basis. Waste haulers periodically deliver grease from grease traps within the City and pump the grease into the digesters. Approximately 230,000 gallons of grease was delivered to the treatment plant in 1998. The primary digesters are mesophilic anaerobic digesters, one with a fixed steel cover and two with floating covers. The floating cover digesters were constructed in 1967. The fixed cover digester was constructed in 1990 and the floating cover digesters were rehabilitated at the same time. The digesters are 45 feet in diameter with a 25-foot side wall depth. The total working volume of the primary digesters with all three in service is 120,000 cubic feet. Mixing is accomplished through confined gas lifting with compressed digester gas and draft tubes. Heating is provided by external heat exchangers. Digester gas is used to produce hot water for digester and control building heating. Excess gas is flared in a waste gas burner.

The digesters have not been cleaned since 1990. Digester gas compressors have had excessive maintenance problems. Equipment suppliers believe that the problem is caused by poor water removal through existing water/gas separators. Plant staff plans to install new water/gas separators.

Capacity

Typical solids loading criteria for anaerobic digestion is 100 to 200 pounds of volatile suspended solids (VSS) per 1,000 ft^3 per day for heated and mixed digesters. A loading rate of 150 pounds of VSS per day per 1,000 cubic feet was used to estimate digester capacity. Total digester capacity at 120,000 cubic feet (all 3 primary digesters in service) is 18,000 pounds of VSS per day based on this criteria. This assumes that all digesters are in service and all digester space is active. Digester capacity is 12,000 pounds of VSS per day with one digester out of service. Projected master plan total solids production ranges from 16,000 lb/day VSS during average loading conditions to 19,000 lb/day VSS during peak 30-day loading conditions. Based on solids production, the digester volume is inadequate for projected master plan solids production.

Digestion capacity is also determined by hydraulic detention time. Minimum hydraulic detention time required for anaerobic digestion is 15 days. Total digester capacity (all 3 primary digesters in service) is about 60,000 gallons per day based on this criteria. Projected master plan total sludge production ranges from 61,000 gallons per day during average loading conditions to 74,000 gallons per day during peak 30-day loading conditions. The actual volume will depend on the thickening performance in the primary sedimentation basins and the DAFT unit. A value of 5 percent was assumed for primary solids and 3.5 percent for WAS. These loadings ignore grease which is about 1 percent of the total digester feed. Based on hydraulic detention time, the digester volume is inadequate for projected master plan solids production.

Deficiencies and Alternative Improvements

Digestion Capacity. The solids digestion system is inadequate for projected master plan solids production. A preliminary review of alternative means of providing digestion capacity was conducted. Improved sludge thickening was not evaluated because thickening would increase the hydraulic detention time but would not reduce the solids loading, which is above the recommended operating criteria.

Available digestion alternatives are continued use of mesophilic digestion with construction of a fourth digester or conversion to thermophilic digestion using extra digester gas and/or waste steam from the Northern California Power Agency power generation facilities.

The existing anaerobic digesters operate under mesophilic temperature conditions of between 97 and 102°F. Thermophilic digestion at approximately 135°F has been reported to provide improved solids dewatering, increased pathogen destruction, and increased scum digestion. In addition, hydraulic detention time may be reduced by a few days in some cases. The disadvantages of thermophilic digestion included higher operating costs, lower process stability, more stringent digester structural and insulation requirements, and additional facilities to provide digester heating. Part of the disadvantages could be overcome by use of waste heat from the adjacent Northern California Power Agency. The facility generally operates continuously during the Summer when peak electrical power is required, but operation may be intermittent during the Winter when electrical power requirements are not at their peak. Given the nature of facility operation, digester heating equipment would be sized to operate without waste heat from the electrical generating facility. Waste heat would reduce the cost of supplemental fuel when the generating facility was operating. Overall, the advantages of using waste heat from the electrical generating facility are not significant given the other disadvantages and cost associated with reconstruction of the digesters and adding digester heating facilities. For these reasons, continued mesophilic digestion is recommended. A fourth digester is needed to provide the required digestion capacity and allow one digester to be taken out of service during off peak periods for maintenance and cleaning.

Digester Feed System. Existing digesters are operated as single-stage units in parallel. With the addition of a fourth digester two-stage operation may provide additional solids stabilization. In a two-stage operation the first three digesters would be operated in parallel and discharge to the fourth digester. The fourth digester would also be heated and mixed to promote digestion but would not be used for gas storage or solids separation as provided for in conventional two-stage digestion. Two-stage digestion as proposed reduces the potential for short circuiting of solids in one digester and can improve volatile solids reduction.

Digester feed is presently switched between digesters automatically. Operational experience with anaerobic sludge digestion systems has shown that optimal performance (*e.g.* volatile solids destruction and gas production) and process stability is obtained when raw sludge is fed to the process continuously and uniformly. An alternative to the present feed system that could provide continuous feed of sludge to the digesters is to use a blending tank for DAFT thickened and primary sludges followed by dedicated feed pumps for each digester. Each digester would then receive an equal proportion of primary and secondary sludge and grease on a continuous basis. The advantages of this alternative are that the process is continuous and relatively simple. The disadvantages are the costs of a blending tank and its mixer, odor control, additional piping, and pumping. The cost of a new blending tank facility is estimated at approximately $700,000.

The continuous feed alternative provides the best process results, but the benefits in plant operation and process would be minor within the Master Plan planning period. Based on cost considerations, the current digester feed scheme should be continued. For a major expansion after Year 2020, continuous feed should be strongly considered as the optimum method of digester feed.

Sludge Grinder. The existing digester piping includes an inline grinder on the lagoon discharge line after digestion. This grinder probably is of little benefit and should be moved to the digester recirculation line. Alternatively, an inline screen may be used to remove rags and other debris from the digester. This would be important if new influent screens were not added to the plant headworks. Assuming new headworks screens are provided then an inline sludge screen is not recommended.

Summary of Recommended Facilities

Recommendations for new facilities associated with sludge stabilization are summarized below:

- Add a fourth anaerobic digester. Operate the digesters in a two-stage mode for enhanced solids stabilization.
- Move the existing inline grinder to the sludge recirculation line. The grinder may not be necessary if headworks screening is added.

DIGESTED SOLIDS HANDLING

Description of Existing Facilities

The overflow from the anaerobic digesters is discharged to Sludge Lagoon No. 2, which is a concrete lined open basin 440 feet long, 110 feet wide, and 9 feet deep. The purpose of this sludge lagoon is to provide sludge storage for periods when application to the land is not possible due to weather, crop management, or other site restrictions. The lagoon also provides additional sludge stabilization. The available storage volume with 2 feet of freeboard is approximately 2.25 million gallons. At a projected Master Plan production of 61,000 gallons of digested sludge per day, the average sludge storage capacity is about 37 days. The actual detention of the sludge is much greater because the solids tend to concentrate at one end of the lagoon as the liquid is pumped from the other end of the lagoon. In addition, lagoon supernatant is pumped to the treated domestic wastewater storage ponds during the Winter to increase lagoon solids storage capacity and detention.

Lagoon contents are periodically applied to the land by pumping to the head of the effluent irrigation system. No portion of the lagoon contents is returned to the main treatment plant processes. A pontoon mounted sludge withdrawal system installed in 1990 was abandoned after operation proved difficult and labor intensive. Once every few years the lagoon water level is lowered sufficiently to allow large equipment to enter the lagoon and push the majority of accumulated sludge toward the pump inlet for removal and application to land.

Sludge Lagoon No. 1 is an unlined earthen basin which is out of service. All digested sludge is discharged to Sludge Lagoon No. 2. There are no bypass provisions and no means to remove Sludge Lagoon No. 2 from service.

The 1976 improvement project included the addition of a belt filter press for digested solids dewatering. The lagoons at that time were intended for emergency storage. The 1990 improvement project lined Sludge Lagoon No. 2 and converted the sludge dewatering building to contain the South Electrical Room and provide miscellaneous storage space. The polymer facilities originally used for sludge dewatering were retained in the building but have been out of service for many years.

Deficiencies and Alternative Improvements

The most significant operating deficiency is the lack of long-term storage capacity and the inability to bypass Sludge Lagoon No. 2 for cleaning. The apparent alternatives are:

1. Pump digested sludge directly to land and bypass Sludge Lagoon No. 2 for a couple of weeks. The digested biosolids could be applied to corn fields, but not to alfalfa.
2. Line Sludge Lagoon No. 1 and place into service.
3. Construct an additional sludge lagoon west of Sludge Lagoon No. 2.
4. Convert the Aerated Pond to a sludge lagoon.

Evaluation of Sludge Storage Alternatives

Bypass Sludge Lagoons When Necessary. Bypass of the sludge lagoon is possible in an emergency, but it is not recommended due to the constraints which would be imposed on the crop irrigation system and farm management. Alternatives which provide additional storage capacity to allow bypass of Sludge Lagoon No. 2 and allow additional storage of solids during the Winter months are preferable.

Sludge Lagoon Expansion. Sludge Lagoon No. 1 could be placed in service with the addition of a concrete liner similar in design to Sludge Lagoon No. 2. Alternative designs include synthetic liners, soil cement, or shotcrete. A second operating lagoon would increase total storage capacity and allow either lagoon to be taken out of service as needed for cleaning or maintenance. The area of Sludge Lagoon No. 1 is about 75 percent of that of Sludge Lagoon No. 1 because of the space taken from Sludge Lagoon No. 1 by the 1990 construction of new secondary clarifiers. The volume of Sludge Lagoon No. 1 after lining is estimated at 1.72 million gallons. The total volume of both lagoons would be slightly over 4 million gallons which would increase the average liquid sludge storage capacity to about 66 days at Master Plan projected conditions. If a liquid decant rate of 30 percent is assumed, this would equal about 94 days of storage capacity. This capacity is less than the recommended process criteria of 120 days. The other problem with improving Sludge Lagoon No. 1 is that it is located in the area needed for long-term expansion of the treatment plant facilities.

New Sludge Lagoon. A new sludge lagoon could be located immediately west of existing Sludge Lagoon No. 2 on land which was previously earmarked for power plant use, but has not been purchased by NCPA. Unlined Sludge Lagoon No. 1 would be abandoned and replaced with a new lined basin with at least 2.8 million gallons storage capacity. This would bring the total storage capacity to approximately 5.2 Mgal. This amount of storage could hold all sludge for 120 days with 30 percent liquid decant. Small surface aerators could be used to dissipate scum and help maintain aerobic conditions at the lagoon surface. The advantage of a new lagoon at this site would be easier integration and operation in conjunction with Sludge Lagoon No. 2. The main disadvantage of this alternative is that future expansion of sludge lagoon capacity and power generation would be limited.

Convert Aerated Pond to Sludge Lagoon. This alternative would include converting the Aerated Pond to a sludge lagoon and abandoning unlined Sludge Lagoon No. 1. The Aerated Pond would provide 4.2 million gallons of additional sludge storage. The Aerated Pond currently contains a buildup of industrial solids and biosolids from previous operation. It is not currently in use. It has an operating depth of 8 feet, which is greater than the existing sludge lagoons. The advantages of converting the Aerated Pond to a sludge storage pond include lower costs, aeration facilities in place, more volume, and better access to feed biosolids into the irrigation system. The main disadvantages of using the Aerated Lagoon are a slight reduction in total available effluent storage and that biosolids may have to be pumped into the pond rather than allowed to flow by gravity. Future expansion of sludge lagoon capacity could be accomplished by converting the Skimming Pond to a sludge lagoon when needed.

Recommended Sludge Lagoon Alternative. Based on operational and future expansion considerations, the recommended alternative is to convert the Aerated Pond to a Sludge Lagoon. This will involve cleaning out the pond, constructing an access ramp, and installing a lining.

Alternative Sludge Drying and Storage Methods

A significant long-term issue is whether the current practice of direct application of liquid sludge to land will be allowed in future discharge requirements. Therefore, alternative dewatering and sludge handling methods are presented as contingencies to the current process. Belt filter press and thermal drying alternatives were discussed in Section 9 and are included again in this discussion of alternatives following paragraphs.

Drying Lagoons. Sludge drying lagoons are a simple variation on the operation of sludge lagoons as practiced at Lodi. A sludge drying lagoon receives digested sludge for a period of approximately 2 years. After the two year period, liquid is decanted off the lagoon and the sludge in the lagoon is allowed to dry. Drying may take one year during dry years or two years during years with relatively high rainfall. The drying sludge is normally turned over several times and then pushed into piles using a front end loader to enhance drying. The cost of drying lagoons may be increased by the need to keep groundwater out of the lagoons through the installation of a lining and/or underdrains. Drying lagoons would be approximately 8 feet deep and occupy approximately 12 acres of land. Groundwater level control facilities would probably also be required to maintain separation between the basin bottoms and groundwater.

Drying Beds. Open drying beds provide dewatering by the process of drainage and evaporation. Sand drying beds historically have been the most common form of this method of dewatering. Other methods include paved beds, wedge wire, and vacuum assisted drying beds. Paved drying beds were selected for comparison to sludge lagoons and mechanical dewatering because they represent an easy to operate, low end technology, and low energy use dewatering method for comparison to mechanical dewatering alternatives.

The principle advantages of paved drying beds over sand drying beds are reduced labor and maintenance costs. Front-end loaders can easily turn the solids to enhance drying and remove dewatered solids from the paved surface. In addition, there is no need to replace sand removed with the biosolids from sand drying beds. The principle disadvantage of paved beds is the lack of an underdrain system for solids draining, although significant drainage may occur with sloped beds and the use of properly designed drain outlets. Solids drying occurs primarily during the warmer months between April and October. At other times storage space is required to hold the biosolids within the paved basin or in the storage lagoons. Approximately 10 acres of paved drying beds would be required at Master Plan design loadings.

Mechanical Dewatering. The three most common mechanical dewatering methods are centrifuges, belt filter presses, and pressure filter presses. Centrifugation applies centrifugal force for the separation of the liquid and solid fractions. The process provides both clarification and compaction. Belt filter presses have moving belts to dewater sludge continuously through a combination of gravity drainage and compression. Pressure filter presses pump sludge through recessed filter plates operated as a batch process which leaves a concentrated sludge cake trapped between the filter plates. All processes have been improved in the last several years and have been the subject of numerous studies. While the selection of a dewatering method is generally site specific and often requires pilot testing, belt filter presses are commonly used for small to medium facilities and would be suitable for Lodi as a mechanical sludge dewatering alternative to drying basins.

Evaluation Of Sludge Drying Alternatives. Drying beds and mechanical dewatering alternatives would need separate liquid sludge storage lagoons for operating flexibility and control of odors. In addition, open drying basins would be needed for the mechanical dewatering alternative to provide dewatered sludge storage for additional drying and storage until final disposal. The estimated costs of sludge drying alternatives are shown in Table 11-8. The capital cost of mechanical dewatering is lowest followed by drying lagoons. The capital costs of paved drying beds are significantly higher than the other alternatives. The operating costs of mechanical dewatering are much greater than the operating cost of drying lagoons or paved drying beds because the drying beds and lagoons require no chemical conditioning, less operational labor, and no special equipment other than a front end loader. From a cost and ease of operation standpoint sludge drying lagoons are the preferred alternative. The primary disadvantage of drying lagoons is the large land area they would require.

Table 11-8. Costs of Sludge Drying Alternatives

Alternative	Capital Cost, dollars	Annual O&M, dollars	Life Cycle Cost, dollars
Drying Lagoons	4,300,000	18,000	4,500,000
Paved Drying Beds	5,000,000	141,000	6,500,000
Mechanical Dewatering	2,700,000	240,000	5,300,000

Biosolids Disposal/Reuse Alternatives

Land application of biosolids is the preferred management alternative assuming that there are no regulatory restrictions imposed on future disposal operations. The potential costs of landfill disposal are presented to demonstrate the impact of eliminating land application of biosolids on City-owned property.

The nearest landfill available for sludge disposal is located between Stockton and Manteca east of Highway 99. This is a Class 2 landfill operated by Forward Inc., a division of Allied Waste Inc. Forward will accept sludge with a solids content of at least 50 percent and not classified as hazardous or restricted material as demonstrated by laboratory testing. Cost of disposal is $28 per ton not including haul costs. Forward estimates haul costs at about $18 per ton based on costs of handling a 20-yard bin. The total cost of disposal is $46 per ton. As much as 2,300 dry tons of solids will be produced per year at projected Master Plan loading conditions. At a solids content of around 60 percent the total weight is 3,900 tons and the cost of disposal is about $180,000 per year. These costs do not include the cost of dewatering which must be operated to produce a dry cake suitable for hauling and disposal. Mechanically dewatered solids must be stockpiled to allow additional drying.

On-site land application of biosolids is the preferred disposal/reuse alternative. Landfill disposal is a backup alternative should land application become prohibited.

Summary of Recommended Facilities

Recommendations for new facilities associated with digested sludge handling are summarized below:

- Relocate the sludge storage facilities away from the existing treatment facilities. Provide a total of three one-acre lined basins. Use the existing sludge storage lagoon and construct two additional lagoons at a site removed from the treatment plant facilities.

- Continue the practice of land-applying biosolids with irrigation water, with distribution improvements as discussed in Section 9. If liquid application of biosolids directly to land becomes no longer feasible or allowed by waste discharge permits, provide sludge dewatering in drying lagoons.

ADVANCED NUTRIENT REMOVAL TREATMENT

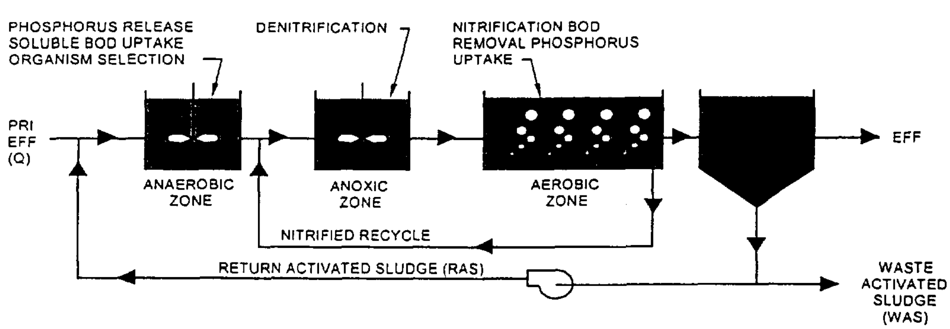
General

Beyond a standard secondary level of treatment with ammonia removal through biological nitrification, additional treatment alternatives include nitrogen and phosphorous removal. These alternatives are presented only to the extent that the master plan facilities provide flexibility to accommodate these and other facilities in the future should they become necessary. The costs of these facilities are compared with other non-conventional treatment or disposal alternatives for meeting potential future discharge requirements later in Section 13.

Nutrient removal processes include a variety of physical/chemical and biological treatment systems. Biological treatment systems are generally less expensive to operate than physical/chemical systems. Integrated systems employ both biological and physical/chemical treatment systems to achieve a greater level of nutrient removal (especially phosphorous) where the physical/chemical treatment systems provide additional removal or backup to the biological systems.

Biological nutrient removal systems are an extension of the conventional activated sludge process for carbonaceous BOD removal. These systems may be adapted for nitrogen or phosphorous removal or both. A basic nutrient removal flow schematic is shown in Figure 11-8. Many flow schemes have been developed for combined nitrogen and phosphorous removal including some patented processes. The final selection of a particular process will depend on the desired level of nutrient removal, current practices at the time of design, and the experience of the process designer.

Figure 11-8. Basic Nutrient Removal Flow



The nutrient removal performance of the existing wastewater treatment process is already substantially better than would be expected because of the fact that decant from the biosolids treatment processes is not returned to the liquid stream treatment process.

Phosphorus Removal

Phosphorous removal occurs through accumulation of phosphorous compounds in micro-organisms at rates 3 to 4 times that of conventional activated sludge systems. An anaerobic selector zone at the beginning of the process usually favors the enhanced removal of phosphorous. This selector zone is typically approximately 10 percent of the total process volume. Phosphorous absorbed by microorganisms is removed by normal sludge wasting. The effectiveness of phosphorous removal will also be dependent on a properly designed and operating final clarifier. Clarifier loading rates may have to be reduced to improve solids capture. Chemical addition to the clarifier or effluent filtration is sometimes required to meet low phosphorous effluent limits. The existing plant is already operating in a mode which accomplishes very good phosphorous removal.

Nitrogen Removal

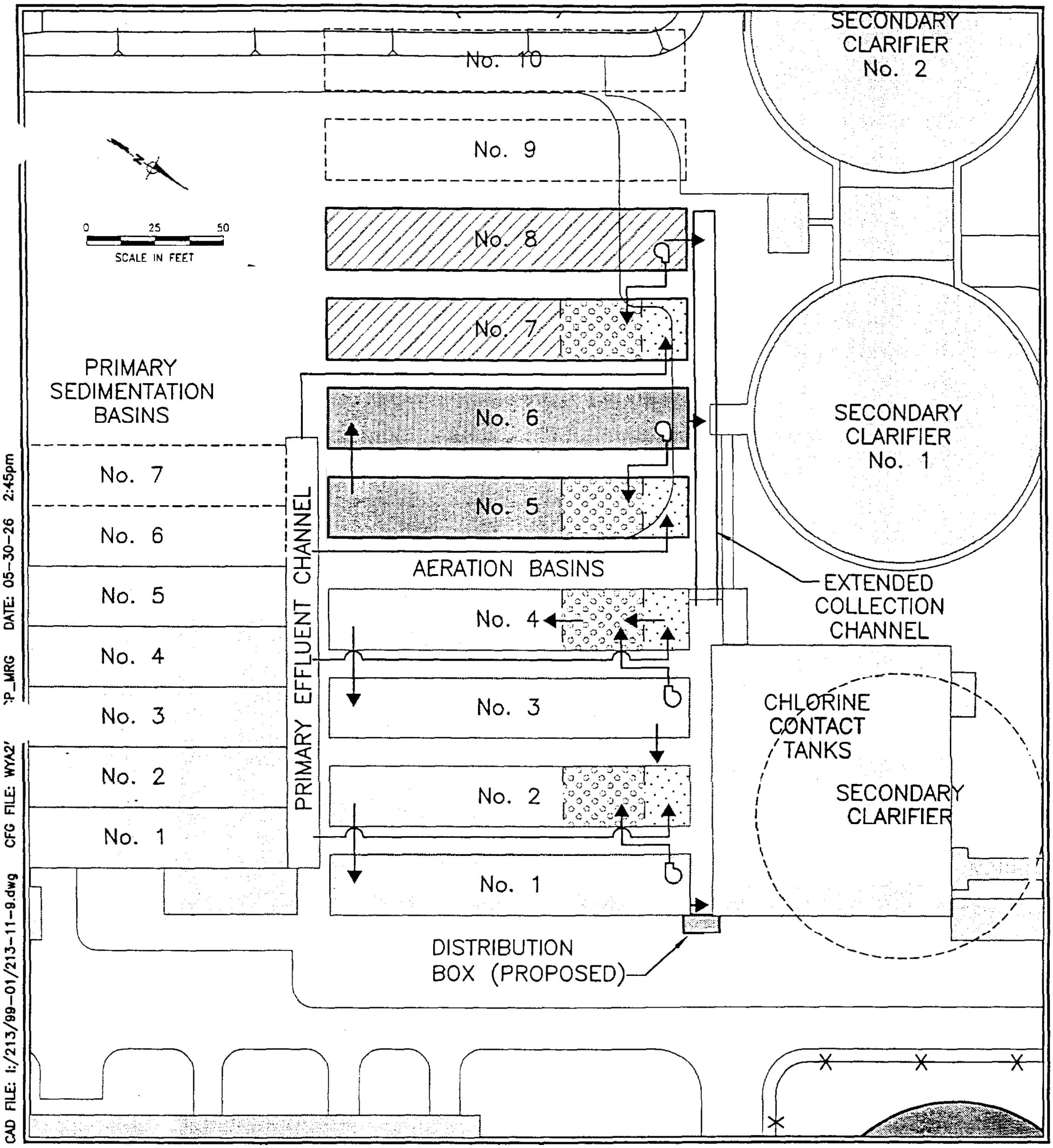
Nitrogen removal occurs through conversion of nitrate to insoluble nitrogen gas in anoxic conditions. This is usually accomplished by having a high amount of RAS recycled to an anoxic zone near the beginning of the process. Nitrogen removal typically requires an increase in tankage volume of 20 to 30 percent compared to the tankage needed for just carbon and ammonia removal. This equates to two additional basins for nitrogen removal assuming six basins are required for carbon oxidation, ammonia nitrification, and incidental biological phosphorous removal at Master Plan projected loading conditions. The partitioning and flow paths for the secondary treatment basins with nitrogen removal are shown in Figure 11-9.

Recommended Facilities Planning

Because the phosphorus removal efficiency is already high, only the facilities required for advanced nitrogen removal need to be addressed at this time. The facilities required to reduce effluent nitrogen concentrations to about 5 mg/L and the estimated costs are listed in Table 11-9. The costs and subjective considerations related to these facilities are compared with the wetlands alternative for denitrification in Section 13—Evaluation of Combined Treatment and Disposal Alternatives.

Table 11-9. Cost of Additional Facilities for Advanced Conventional Nitrogen Removal

Capital Cost, dollars	Operation and Maintenance, dollars	Life Cycle, dollars
5,800,000	100,000	6,900,000



Legend:

- Existing Facility
- Master Plan Facility
- Master Plan Plus Denitrification
- Anoxic Zone
- Anaerobic Zone
- Long-term Facility
- Baffle
- Flow Direction
- Mixed Liquor Recycle Pump

Figure 11-9

City of Lodi
Water Pollution Control Facility

AERATION BASINS WITH NITROGEN REMOVAL FACILITIES

ADVANCED HEAVY METALS REMOVAL TREATMENT

Advanced Solids Removal

Some of the total recoverable metals are bound with particulates in secondary effluent. Removal of particulate bound metals can help assure compliance with discharge requirements since the discharge requirements currently are applied to total metals with a translator factor to conservatively estimate maximum dissolved metals. Advanced solids removal using tertiary treatment (coagulation and effluent filtration) could remove over half the effluent TSS and associated metals. This would not be as effective as treatment technologies which remove dissolved metals, but it may be sufficient to meet discharge requirements. Costs associated with tertiary treatment are presented in the next sub-section of this report.

Chemical precipitation is sometimes used to remove dissolved metals from industrial wastewater[1]. The major disadvantages of metals removal through chemical precipitation are the ongoing costs of the chemicals and the treatment and disposal of additional sludge. Hydroxide and sulfide are used for direct precipitation, while alum or iron flocs are effective at coprecipitating some metals. Metals removal as a function of pH is shown in Figure 11.10. As can be seen in the figure, the best zinc removal at an optimal pH would leave a concentration of about 100 Mg/L with hydroxide precipitation. This is almost the same as the discharge limit, so it would not be reliable enough as a process. Sulfide precipitation would produce low enough metals concentration to meet discharge limits as long as the pH could be maintained above about 5.

Sulfide precipitation occurs naturally to a degree in the influent sewer line and primary treatment because of the presence of hydrogen sulfide. This may reduce dissolved zinc and lead, leaving most of the remaining metals in the particulate form leaving the primary clarifier. Some of the metals could possibly resolubilize in the secondary treatment process. The addition of sulfide to the secondary clarifiers for final treatment would again remove soluble metals. However, this would leave excess sulfide in the effluent prior to chlorination and would dramatically increase chlorine usage. It would also require special odor and corrosion control measures. Because of these issues, sulfide precipitation is not practiced on secondary effluent at any known treatment plant. Therefore, removal of dissolved metals by sulfide precipitation is not recommended as a viable alternative for Lodi.

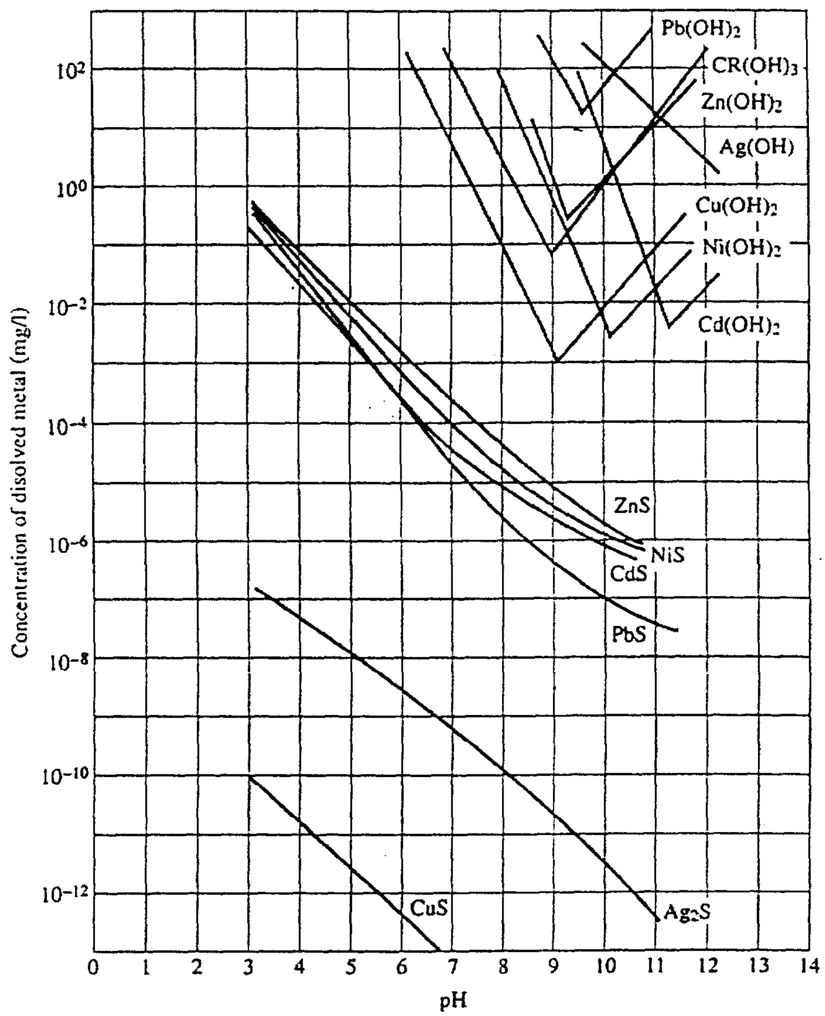
Other Advanced Metals Removal Processes

There are other processes for advanced metals removal including ion exchange and reverse osmosis. These other alternatives are more expensive than chemical precipitation and would only be worth consideration if there were no other alternatives for satisfying metals standards in discharge requirements.

Recommended Heavy Metals Removal Treatment

If metals removed must be performed using conventional treatment processes, coagulation and effluent filtration are recommended. Pilot testing should be performed to determine if adequate

Figure 11-10. Approximate Best Achievable Effluent Heavy Metal Concentrations for Lime and Sulfide Precipitation[1]



removal can be achieved. Effluent filtration is compared with wetlands for metals removal in Section 13

EFFLUENT FILTRATION

Effluent coagulation and filtration are required for most of the surface discharge alternatives. It would provide enhanced BOD removal and may provide additional removal of some heavy metals. If constructed, the proposed Sports Complex would be irrigated with reclaimed water. In a previous study prepared by HYA Consulting Engineers in 1995, a tertiary filtration and disinfection capacity of 2.5 Mgd was recommended to meet irrigation needs. The water reclamation plant was proposed for a location east of the existing chlorine contact tank approximately where the third secondary clarifier is proposed in this report. A copy of the HYA report is contained in Appendix [redacted].

There are several types of filtration systems that can be used for the effluent filtration application. The three systems evaluated for master planning purposes were:

1. Granular Media Filtration
2. Sỹnthetic Media (Fuzzy) Filters
3. Membrane Filtration

Granular Media Filtration

Granular media filtration has been the industry standard filtration technology for several decades. Granular media filters have been improved to provide greater effective use of the entire media bed depth to trap and hold particles. Two of the most popular designs have been pulsed bed (ex. Hydroclear) and continuous cleaned upflow filters (*i.e.* Parkson Dynasand). The typical design hydraulic loading rate for advanced granular media filters is 5 gpm/sq ft for tertiary reclamation treatment.

Synthetic Media (Fuzzy) Filters

Fuzzy Filters are a depth filter like a granular media filter, but with a special compressible filament media bed. The Fuzzy Filter media has much greater void space than granular media filters, allowing design unit area loading rates to be in the range of 20 to 25 gpm/sq ft The media compression can be adjusted to provide the desired performance. Fuzzy Filters also require much less backwash water than pulsed bed or some of the other granular media filters. There is currently only one small commercial Fuzzy Filter installation in California producing tertiary reclaimed water. The California Department of Health Services has yet to fully approve of the use of Fuzzy Filters operated at high hydraulic loading rates for producing reclaimed water.

Membrane Microfiltration

Membrane microfiltration technology has been improving rapidly to the point where it is becoming more competitive with granular media filtration. One of the advantages of membrane microfiltration is that it provides better direct removal of bacteria, viruses, and free swimming parasites than granular filtration. The main disadvantages for membrane filters are that costs are significantly higher than for granular media filters and there are few installations in California for producing tertiary reclaimed water. Membrane microfiltration has proven to be a cost effective pretreatment step prior to reverse osmosis of effluent. Performance requirements in terms of turbidity are over 10 times more stringent for membrane filters than for granular media filters (0.5 NTU max. versus 10.0 NTU max., respectively).

Evaluation of Filtration Alternatives and Recommended Alternative

Cost estimates for the filtration alternatives are shown in Table 11-10. It should be noted that the costs shown are for the filters only and do not include the costs of related facilities required for tertiary treatment. The costs of the Fuzzy Filter facilities and the membrane filters were partially based on a study performed for Dublin-San Ramon Services District[2]. Because of the lower costs and proven track record for granular media filtration, it is the recommended alternative at this time. The proposed location for effluent filters was shown on Figure 11-4. The cost of media for the Fuzzy Filters is expected to gradually drop over the next few years due to manufacturing efficiency improvements. Membrane filter costs are also expected to become more competitive in the future. Both fuzzy filters and membrane filters should be reevaluated in any future predesign of tertiary filtration facilities. The costs of complete tertiary filtration and disinfection facilities are compared with other non-conventional treatment or disposal alternatives for meeting potential future discharge requirements later in Section 13. The impact of constructing tertiary treatment facilities for only 2.5 Mgd for the proposed sports complex is also addressed in Section 13.

Table 11-10. Costs of Filtration Alternatives[a]

Alternative	Capital Cost, dollars	Annual O&M, dollars	Life Cycle Cost, dollars
Granular Media	7,500,000	170,000	9,300,000
Synthetic Medium	6,500,000	170,000	8,300,000
Membrane Microfiltration	17,000,000	1,000,000	27,800,000

(a) Costs of filtration facilities only, not complete tertiary treatment facilities.

PLANT SUPPORT SYSTEMS

Other general facilities improvements which are desirable because of safety, operational, or reliability considerations have also been evaluated, including existing buildings, electrical service, and control and monitoring systems.

Administration Space and Personnel Spaces (Washrooms and Lockers)

Buildings at the treatment plant house four general functions: General office space and operations areas including locker space and showers, laboratory space for analytical work associated with process control and monitoring, plant and utilities maintenance, and process space for chemical storage, handling, and equipment.

The Control Building houses the plant operations staff, plant process monitoring room, laboratory, lunch room/meeting room, men's and women's shower/locker rooms, blower room, maintenance shop, and miscellaneous storage rooms.

There are up to 18 people on the day shift, with Thursday having the largest regularly scheduled staff size at 17. The numbers of each staff classification are as follows:

- 6 Operators
- 2 Mechanics
- 1 Maintenance Person

- 1 Electrician
- 1 Assistant Superintendent
- 1 Chief Operator
- 1 Lab Supervisor
- 2 Laboratory Technicians
- 2 Inspectors
- 1 Administration Clerk

Two staff positions will probably be added for the expansion of the secondary treatment facilities and solids treatment improvements. If tertiary treatment is constructed at the plant, another additional 3 staff positions would probably be required.

The laboratory space is currently adequate, but there is no room for expansion. The maintenance shop size is just adequate. The main men's locker room could accommodate two or three extra people.

Deficiencies

After visiting the site and discussing space requirements with key plant staff, five deficiencies were identified:

1. Laboratory staff has indicated that a window would enhance their work space.
2. The Chief Operator has no enclosed office area for private conversation.
3. Women's Restroom and Shower—There is only one lavatory, which is enclosed in the accessible toilet compartment. Calculations of required toilets based on Appendix Chapter 29 of the California Building Code show that a minimum of two toilets are required for men and women. There is currently only one toilet for women.
4. Women's Locker Room—The women's locker space within the Control Building is inadequate and should be expanded to provide additional lockers and washrooms separate from the toilet facilities.
5. Maintenance Vehicles Parking—The maintenance shop in addition to providing space for maintenance activities is used for parking vehicles after normal work hours. A separate garage is needed to protect equipment and small utility vehicles and allow expansion of maintenance activities within the maintenance shop.

Recommended Improvements

- The laboratory has exterior walls on two sides, and the addition of windows would be relatively simple. Windows matching the existing could be installed, with opening vents. As an alternative, glass block could be installed in the existing concrete block walls, giving a view of the outside but not adding the problems of open windows altering the heating, cooling and fume hood air flows.

- By relocating the clothes washer and dryer from the existing utility room to the room presently occupied by the Maintenance Superintendent, accessible showers can be installed in that room for the women staff. In addition, the existing shower can be converted to the required additional toilet, and one or two lavatories can be installed adjacent to the new toilet.

- The Senior Plant Mechanic's office can be relocated to the second floor adjacent to the air handling units. The space is currently unused, but is served only by a spiral staircase located inside the workshop. A new exterior stair is proposed on the west side of the building, outside the current Maintenance office. To accommodate the new office, the exterior louvers will be removed from the area directly outside the room, and a door and windows will be installed. The wall separating the office from the air handling units will be replaced with an acoustically separating wall (gypsum board on acoustical channels on studs, with sound insulation between studs). An acoustical ceiling will be installed in the office, with insulation above the ceiling to stop sound coming over the top of the wall.

- The unused concrete pad north of the digesters near Sludge Lagoon No. 1 should be converted to Maintenance parking.

There is no easy way to provide the Chief Operator with an enclosed office area. It might be possible to add two walls to the existing area used by the Chief Operator. Recommended improvements to the Control Building and maintenance parking are shown in Figure 11-11.

Electrical Service

The plant has dual 2,000 Amp, 480V, 3-phase utility power feeds and no standby power generators. The south feed is PG&E Meter No. 29159 and the north feed is SMUD Meter No. 29467. The common point of power interconnectivity is at the main switchboard and the bus tie transfer switch. This has proved reliable except during the most extreme conditions where power failures have effected the entire West Coast. The City has considered an additional power feed to the Northern California Power Agency (NCPA) electrical generating facility located behind the treatment plant. This facility uses reclaimed water from the Water Pollution Control Plant for steam generation and cooling. As a result, the NCPA facility is dependent on the treatment plant for continued operation.

Given the lack of any data to suggest that the power supply is inadequate, it does not appear reasonable at this time to add standby power facilities. However, as treatment requirements become more stringent or wastewater reuse involves public contact, the regulatory requirements for standby power must be addressed. An evaluation of power supply alternatives is beyond the scope of the master plan. This evaluation should be provided as part of the predesign of the treatment plant improvements.

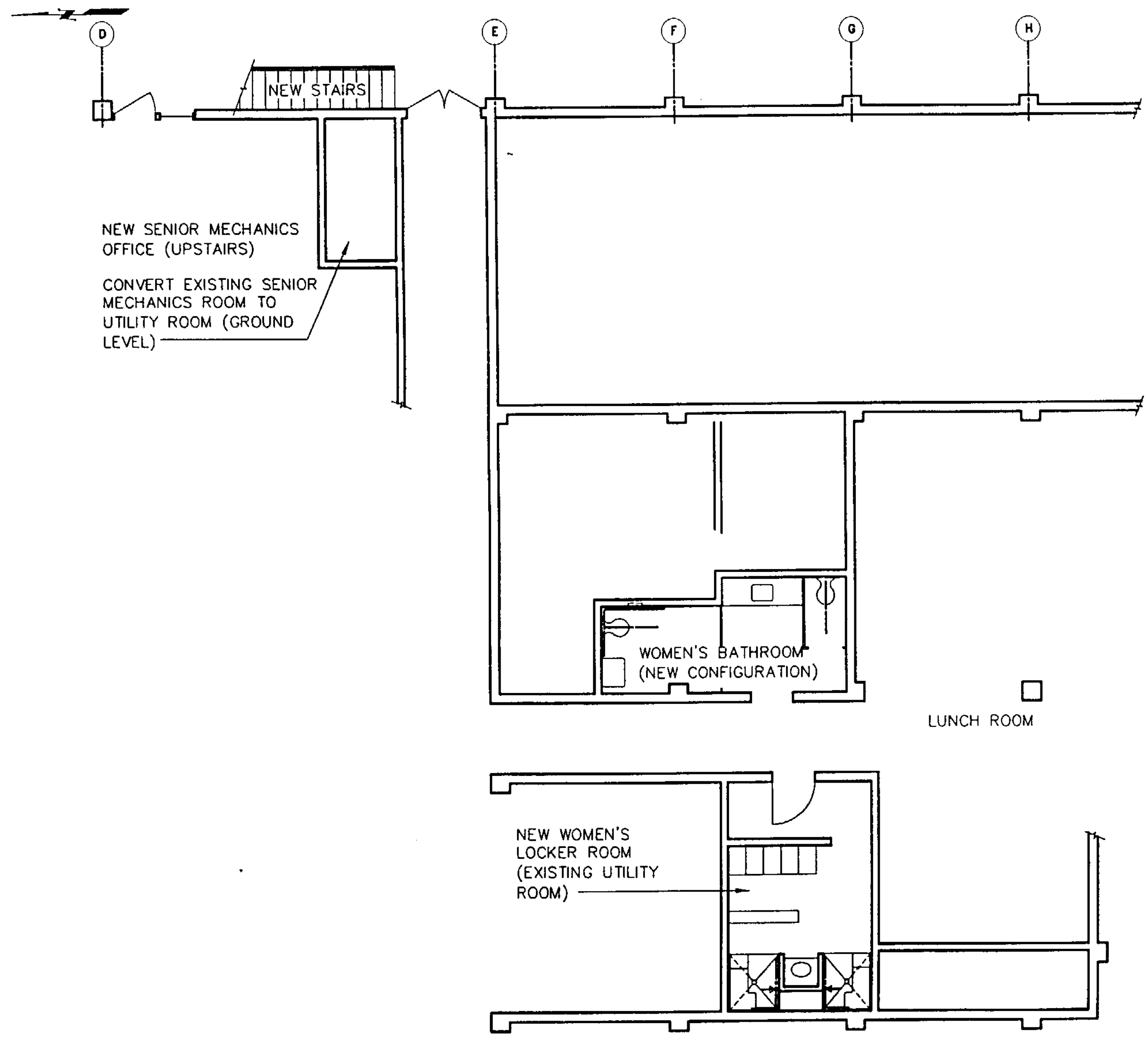


Figure 11-11

City of Lodi
Water Pollution Control Facility

Recommended Control Building Improvements

SCALE IN FEET

NOTES:

LEGEND:

WEST YOST & ASSOCIATES

The addition of new influent pumps, aeration blowers, and other facilities will exceed the current capacity of the main switchgear. Preliminary calculations show that the maximum running loads could be as high as 2,200 amps on a service rated at 2,000 amps. Running loads would be greater if effluent filtration or other processes are added. In addition, distribution panels DP-B1 and DP-B2, which are rated at 1,600 amps, will have to be replaced or supplemented with new electrical equipment.

Flood Protection

FEMA flood insurance rate maps show the 100-year flood elevation for the treatment plant area at elevation 8 feet based on the 1929 National Geodetic Vertical Datum. The treatment plant site is at elevation 9 feet or above based on an unknown datum. Entrance to structures is located at elevation 10 feet or above. Storm water on the treatment plant site sheet flows across the site to surrounding ditches and sloughs.

Flood protection for the plant appears adequate to maintain operations during the 100-year storm event. The basis of treatment plant elevations and reference datum should be identified in relation to the 1929 datum in order to confirm actual elevation differences between existing treatment plant elevations and flood elevations.

SUMMARY OF TREATMENT PLANT IMPROVEMENT RECOMMENDATIONS

Some treatment plant deficiencies could be addressed by several alternatives. The comparison of alternative improvements discussed in this section is summarized in Table 11-11. All recommended treatment plant improvements needed over the 20 year planning period are summarized in Table 11-12. Estimated total project costs of the facilities and improvements needed by the year 2004 are shown in Table 11-13. Some of the treatment process alternatives must be compared with other alternatives not related to treatment plant improvements. These comparisons are performed later in Section 13.

Table 11-11. Treatment Plant Improvement Alternatives

Process	Alternative Improvements	Major Subjective Advantages	Major Subjective Disadvantages	Capital Cost, dollars	Life Cycle Cost
Influent Screening	Mechanically cleaned bar screens	Only one submerged moving part	Large openings High clearance requirement	510,000	n/a
	Continuous self-cleaning screens[a]	Finer screen High loading capacity Proven	Need separate washing and compacting	650,000	n/a
	Spiral basket screens[b]	Combined screening, washing, and compacting	Limited capacity	690,000	n/a
Grit Removal	**Aerated grit tanks**	Preaeration	Larger facility	1,900,000	2,000,000
	Vortex grit removal system	Lower energy use Compact	Grit hopper plugs more often	1,200,000	1,300,000
Disinfection	Gas system containment, scrubbing	Fewer truck deliveries	Safety Chlorination byproducts	1,400,000	2,800,000
	Liquid hypochlorite and bisulfite (outside suppliers)	Simple and safe	More truck deliveries Chlorination byproducts	260,000	3,400,000
	On-site liquid hypochlorite generation	Fewer truck deliveries	Complexity Increased maintenance Chlorination byproducts	1,100,000	3,900,000
	Ultraviolet disinfection	No chlorination byproducts No chemical deliveries or handling Better virus kill	Less reliable disinfection w/o filtration High maintenance	4,800,000	7,700,000
Sludge Lagoons	Line Sludge Lagoon #1		Easier to implement	430,000	n/a
	One new lagoon, abandon existing Lagoon #1	Leaves room for plant expansion Moves potential odor sources west	Requires more land	1,200,000	n/a
	Convert aerated pond	Most capacity Closest to Irrigation Boxes	Requires digested sludge pumping	930,000	n/a
Sludge Dewatering	**Drying lagoons**	Simple	Requires more land	4,300,000	4,500,000
	Paved drying beds		Moderate labor	5,000,000	6,500,000
	Mechanical dewatering	Low land requirements	High labor High chemical usage	2,800,000	5,300,000
Effluent Filtration	**Granular media filtration**	Proven		8,000,000	9,800,000
	Membrane filtration	Fewer particle breakthroughs		17,000,000	27,800,000
	Fuzzy filters	Compact		6,500,000	8,300,000

[a] Alternatives shown in **bold** are preferred.
[b] Alternatives shown in *italics* should be reevaluated for treatment plant upgrades after Year 2005 because of potential improvements in technology.

Table 11-12. Summary of All Treatment Plant Improvement Recommendations

Process	Deficiencies	Recommended Improvements	Flow at Upgrade	Year
Headworks	Capacity	Modify existing headworks Construct new headworks	7 8.5	2004 2020
Screening	No screening - only comminuting	Continuous self-cleaning screens	7	2004
Grit Removal	Marginal performance, no room for expansion	Aerated grit tanks *Or Vortex grit removal system*	8.5	2020
Cross Channel Gates	Capacity	Repair gates	6.5	Immediate
Septage Receiving Station	Capacity, plugging	Upgrade existing for less plugging, easier maintenance Construct new station	7 8	2004 2015
Domestic Influent Pumps	Capacity	Install larger pumps or impellers, or increase speed	7	2004
Industrial Influent Pumps	Capacity, redundancy	Upgrade one pump, install a small third pump		Immediate
Primary Clarifiers	Capacity	Two new clarifiers	9.5	>2020
Aeration Basins	Capacity Treatment level	Upgrade diffusers (completed 12/99) Two new aeration basins Three more aeration basins (to 12 Mgd capacity)	6.5 7 8.5	Immediate 2004 2020
Aeration Basins	Process stability	Replace diffusers with mixers	7	2004
Secondary Clarifiers	Capacity, redundancy	Flocculent injection system Construct a third clarifier	6.5 7	Immediate 2004
Blowers	Capacity	Two new blowers	7	2004
Blower Control	Efficiency - not demand-matching	Automate, including air manifold valves	7	2004
RAS Pumps	Excessive capacity	Trim impellers	6.5	Immediate
WAS Pumps	Excessive capacity	Downsize or trim impellers for 2 pumps	6.5	Immediate
Disinfection	Safety Improved reliability	Liquid hypochlorite and bisulfite (outside suppliers) Enhanced CCT flushing	7 7	2004 2004
WAS Thickening	Capacity Redundancy	Continuous operation, restore polymer system Construct second DAFT	6.5 7	Immediate 2004
DAFT Subnatant	No disinfection	Return subnatant to headworks	7	2004
Anaerobic Digestion	Capacity, redundancy	Construct fourth digester Construct fifth digester	7 8.5	2004 2020

Process	Deficiencies	Recommended Improvements	Flow at Upgrade	Year
Sludge Grinder	Low effectiveness	Move to sludge recirculation line	7	2004
Sludge Lagoons	Storage capacity, redundancy	Convert aerated pond to sludge lagoon, abandon unlined Sludge Lagoon No. 1	7	2004
Sludge Dewatering	Compliance with possible future discharge requirements	Drying lagoons	N/A	If & when required
Nitrogen Removal	Compliance with possible future discharge requirements	Two additional basins, denitrification compartments, mixed liquor recycle	N/A	See Sec. 13
Advanced Heavy Metals Removal	Compliance with possible future discharge requirements	Chemical precipitation	N/A	See Sec. 13
Tertiary Filtration	Compliance with possible future discharge requirements	Granular media filtration *Or Fuzzy filters*	N/A	See Sec. 13
Maintenance Shop	Parking	Construct parking garage	7	2004
Maintenance Supervisor's Office	Space	Move upstairs	7	2004
Women's Locker Room	Space, lockers	Expand	6.5	Immediate
Control System	Manual control adjustment, no SCADA	Install centralized automated control system with manual override features	7	2004

(a) Improvements shown in *italics* are alternatives which should be reevaluated for treatment plant upgrades after Year 2005 because of potential improvements in technology.

Table 11-13. Estimated Costs of Treatment Plant Improvements Common to All Process Alternatives [a]

Item	Estimated Capital Cost, dollars[b]
Provide Additional Capacity	
Modify Domestic Pumps	210,000[c]
Replace Industrial Pump, add 3rd	360,000[c]
WAS Thickening Improvements	1,540,000[c]
New Anaerobic Digester	1,710,000 [c]
Sludge Lagoon Capacity	930,000 [c]
Subtotal	4,750,000
Comply with Regulatory Requirements	
2 New Aeration Basins	4,440,000 [d]
New Secondary Clarifier	2,700,000 [d]
RAS/WAS Improvements	1,360,000 [d]
Liquid Hypochlorite, Bisulfite	260,000 [d]
Storage Pond Aeration	250,000 [d]
Operations Building Improvements	150,000 [d]
Odor Control Facilities	890,000 [d]
Subtotal	10,050,000
Rehabilitation & Operations Improvements	
Headworks Improvements	260,000
Influent Screening	650,000
Septage Receiving Station	60,000
Clarifier Polymer Feed System	290,000
Control System Upgrades	319,700
Miscellaneous Operational Upgrades	140,300
Subtotal	1,720,000
Common Improvements Total	16,520,000
Value of Previously Deferred Components	4,390,000

(a) Treatment plant costs associated with individual alternatives not included in this table.

(b) All Costs shown include escalation to an ENR index of 7000, plus allowances for engineering, administration and contingencies.

(c) Component deleted from last plant expansion project.

(d) Facilities required, in part, to restore capacity lost when operations were adjusted to nitrify ammonia in response to changes in discharge requirements.

REFERENCES

1 W.W. Eckenfelder, Jr. *Industrial Water Pollution Control.* Third Edition, McGraw-Hill, 2000.

2 Whitley Burchett and Associates. *Granular-Media Filtration, Synthetic-Medium Filtration, and Microfiltration for Wastewater Reclamation: A Case Study.* CWEA Annual Conference, April 2000.

SECTION 12. ODOR CONTROL

INTRODUCTION

The current odor situation at the Lodi Plant was evaluated through site visits, assessment of existing data, and discussion with City staff. The plant receives relatively few formal odor complaints; however, many people are aware of the facility because of its visible location along the I-5 freeway. With increased development in the vicinity of the plant in the future, improved odor control will become necessary.

OVERVIEW OF ODOR SOURCES

Strong odors are emitted from the plant at times. Based on existing information, the factors listed below are the primary odor sources at the plant (in approximate order of importance/priority).

Influent Sewers/Headworks

Historical information indicates that both the domestic and industrial sewers contain high-strength odor at times which is released at the entrance to the plant and at the headworks facility. The industrial sewer has peak emissions during the canning season period, however, the domestic sewer has more consistently strong odor through most months of the year. Ventilation from the headworks emits influent odors directly to the atmosphere.

Grit Washing and Storage

In summer especially, the grit washing and loadout area is very odorous due to high sulfide and other reduced sulfur and nitrogen compounds.

Primary Sedimentation Basins

The primary sedimentation basins (PSBs) are odorous at areas where there is turbulence, *i.e.* inlet channels, outlet channels, and particularly at the drop weirs, but also at the scum drawoff points. The quiescent portion of the PSBs does not appear to have major odor emissions. The weir area has by far the greatest turbulence, and, therefore, the greatest odor emissions from the PSBs.

Septage Discharge

Significant peak odor events occur at the headworks and to some extent at the primary PSBs whenever a load of septage is discharged to the influent sewer.

Grease Truck Unloading and Scum Sump

Trucked grease is unloaded directly to this sump beside the PSBs and fed directly to the digesters. This unloading and sump operation creates a very strong localized odor, especially in the warmer months.

Digesters

There are some odor emissions from the annular spaces on the two floating cover digesters and exposed sludge at overflow points. There can also be digester gas leaks at gas release valves or other gas leakage at times.

Digester Gas Combustion

Digester gas contains high-strength odorants, particularly hydrogen sulfide and other reduced sulfur compounds. Current hydrogen sulfide concentrations in the raw digester gas at Lodi are between 1,000 and 2,000 ppm. Therefore, any leakage of raw gas presents a serious odor emission. The sulfur compounds are largely oxidized to sulfur dioxide during combustion and sulfur dioxide also has a distinctive odor. Therefore, it is desirable to keep the hydrogen sulfide content in digester gas to a minimum. There are also corrosion and related problems when burning digester gas with high hydrogen sulfide content. New air emission rules require Lodi to burn only gas which has less than 200 ppm hydrogen sulfide. An iron sponge system was installed 10 years ago to extend the life of the boilers. However, if this unit is fed the current high concentrations, the media life in the iron sponge system is greatly shortened.

Sludge Lagoon No. 2

Under most conditions, the lagoon is probably not a significant odor source at the plant. However, when it is being cleaned or sludge is being removed, it may present more of a problem.

Dissolved Air Flotation Thickener (DAFT)

The DAFT can be odorous at times, especially if fed waste activated sludge (WAS) that has gone septic due to short-term storage. DAFTs are best fed fresh WAS on a continuous basis to minimize odor emissions.

Land Application Sites

At certain times of the year and under certain conditions there can be significant odor from the wastewater application fields near the plant. Also, stabilized biosolids, when applied to the nearby agricultural fields, can be odorous if not applied and managed properly. Odors from land application of wastewater and biosolids are minimized through proper loading rates and operational practices as discussed previously in Sections 8, 9 and 10.

Other sources at the plant are not considered major odor sources. However, under unusual conditions, the effluent ponds or other sources may be noticeable downwind. Once the primary odor sources at the plant are controlled, the remaining sources can be better evaluated to determine the extent of their impact and define further odor controls that could be required. The emphasis in this section is on controlling odor at the major sources—the influent sewers, headworks, trucked waste discharges, PSBs, and solids processing odors. The land application facilities and associated odor control needs were discussed in Sections 8 and 9.

WASTEWATER ODORANTS

Most odorous wastewater substances can be categorized as reduced or partially-reduced compounds. Hydrogen sulfide (H_2S), which is a common sewer and wastewater gas, is an inorganic reduced sulfur compound. Odorous organic reduced compounds include mercaptans and organic sulfides such as dimethyl sulfide. Other compounds, such as aldehydes, ketones, and carboxylic acids (all non-hazardous), usually come from industrial discharge or from the organics in domestic wastewater. In general, odorous compounds are the result of biological activity, such as anaerobic decomposition of organic matter containing sulfur and nitrogen. Aerobic decomposition or solids-processing which involves heating the sludge can contribute to the production of odorous compounds as well. Table 12-1 presents characteristics of common odorants at wastewater treatment plants.

Table 12-1. Common Odorous Compounds in Wastewater

Compound Name	Formula	Molecular Weight	Volatility at 25°C, ppm (v/v)	Detection Threshold, ppm (v/v)	Recognition Threshold, ppm (v/v)	Odor Description
Acetaldehyde	CH_3CHO	44	Gas	0.067	0.21	Pungent, fruity
Allyl mercaptan	CH_2:CHC H_2SH	74		0.0001	0.00015	Disagreeable, garlic
Ammonia	NH_3	17	Gas	17	37	Pungent, irritating
Amyl mercaptan	$CH_3(CH_2)_4SH$	104		0.0003	—	Unpleasant, putrid
Benzyl mercaptan	$C_6H_5CH_2SH$	124		0.0002	0.00026	Unpleasant, strong
n-Butyl amine	$CH_3(CH_2)NH_2$	73	93,000	0.080	1.8	Sour, ammonia
Chlorine	Cl_2	71	Gas	0.080	0.31	Pungent, suffociating
Dibutyl amine	$(C_4H_9)_2NH$	129	8,000	0.016	—	Fishy
Diisopropyl amine	$(C_3H_7)_2NH$	101		0.13	0.38	Fishy
Dimethyl amine	$(CH_3)_2NH$	45	Gas	0.34	—	Putrid, fishy
Dimethyl sulfide	$(CH_3)_2S$	62	830,000	0.001	0.001	Decayed cabbage
Dyphenyl sulfide	$(C_6H_5)_2S$	186	100	0.0001	0.0021	Unpleasant
Ethyl amine	$C_2H_5NH_2$	45	Gas	0.27	1.7	Ammonialike
Ethyl mercaptan	C_2H_5SH	62	710,000	0.0003	0.0001	Decayed cabbage
Hydrogen sulfide	H_2S	34	Gas	0.0005	0.0047	Rotten eggs
Indole	$C_6H_4(CH)_2NH$	117	360	0.0001	—	Fecal, nauseating
Methyl amine	CH_3NH_2	31	Gas	4.7	—	Putrid, fishy
Methyl mercaptan	CH_3SH	48	Gas	0.0005	0.0010	Rotten cabbage
Ozone	O_3	48	Gas	0.5	—	Pungent irritating
Phenyl mercaptan	C_6H_5SH	110	2,000	0.0003	0.0015	Putrid, garlic
Propyl mercaptan	C_3H_7SH	76	220,000	0.0005	0.020	Unpleasant
Pyridine	C_5H_5N	79	27,000	0.66	0.74	Pungent, irritating
Skatole	C_9H_9N	131	200	0.001	0.050	Fecal, nauseating

Compound Name	Formula	Molecular Weight	Volatility at 25°C, ppm (v/v)	Detection Threshold, ppm (v/v)	Recognition Threshold, ppm (v/v)	Odor Description
Sulfur dioxide	SO_2	64	Gas	2.7	4.4	Pungent, irritating
Thiocresol	$CH_3C_6H_4SH$	124		0.0001	—	Skunky, irritating
Trimethyl amine	$(CH_3)_3N$	59	Gas	0.0004	—	Pungent, fishy

Source: Water Environment Federation, 1995

In general, the lower the molecular weight of the odorous compound, the higher its vapor pressure and its potential to transfer from the wastewater to the atmosphere. In theory, Henry's Law governs the mass transfer of odorous gases from the liquid phase to the gaseous phase. Henry's Law states that the partial pressure of the gas above liquid surface is directly proportional to the molecular concentration of the gas dissolved in liquid. In practice, wastewater pH and temperature, the rate of each odorant generation within a system, the turbulence of the wastewater stream, and air ventilation rate determine the odor concentration in the atmosphere.

PLANT INFLUENT CHARACTERISTICS AND HEADWORKS SITUATION

Considerable historical data are available on the plant influent characteristics. On the domestic sewer, BOD and SS are medium strength, but total sulfide concentration varies widely – from zero up to about 8 mg/L, with an average of about 4 mg/L (CDM, 1999). The plant influent pH can often be depressed below 7.0 (down to about 6.5 at times), particularly in long dry weather periods, indicating substantial anaerobic activity is occurring upstream in the interceptor system. Wastewater temperature is relatively warm in the spring through fall months (about 23 to 30°C). These wastewater characteristics are conducive to high-strength odors, particularly reduced sulfur compounds.

Gas phase hydrogen sulfide measurements from the influent domestic sewer are often between 2 and 20 ppm. Occasionally, hydrogen sulfide concentrations are probably well above this range. Corrosion of influent concrete pipe has been verified on the domestic sewer, and there is also evidence of concrete and metal corrosion within the headworks, particularly in or adjacent to the domestic influent channels. This corrosion is caused by sulfuric acid which forms when hydrogen sulfide is absorbed onto moist surfaces where microorganisms convert hydrogen sulfide to sulfuric acid.

The industrial sewer has its peak flows during the canning and food processing seasons (late summer and early fall). The industrial flow at these times contains high BOD, causing anaerobic conditions in the industrial wastewater and production of large quantities of reduced sulfur compounds. At times, this sewer can create very strong odor emissions at and prior to the headworks.

There are also critical worker safety considerations when dealing with influent flows having the above characteristics. Toxic gases can accumulate in these interceptor sewers because of inadvertent discharge of chemicals upstream in the collection system, hydrogen sulfide off-gassing from the wastewater (especially at low pH), or other cause. For this reason, ventilation of influent sewers and occupied headworks rooms (particularly below-grade rooms) becomes an important issue.

The plant recently modified the ventilation in this area to prevent or greatly minimize toxic gas alarms in the headworks room. Ventilation of the inlet sewers just upstream of the headworks is now provided so that this sewer foul air is prevented from entering the headworks room (reported to be 2,000 cfm of ventilation). Headworks room ventilation was increased to 7,000 cfm, with an air supply fan providing this amount of fresh supply air, and two exhaust fans at 3,500 cfm each removing foul air from the room. The exhaust fans discharge the foul air directly to atmosphere above the headworks room. This modification has improved the atmosphere in the headworks room.

CHEMICAL ADDITION

The plant currently utilizes no chemical addition to the influent trunk sewer to help control odor or corrosion. Using chemicals for this purpose is now a common practice at California wastewater treatment plants, and is normally cost-effective for plants with relatively high influent sulfide levels. The three most commonly used chemicals are hydrogen peroxide, sodium hypochlorite, and iron chloride. Masking agents are sometimes considered for odor control, but are generally not as effective as oxidants or sulfide binding chemicals. The plant also does not utilize iron addition to digesters for control of hydrogen sulfide in the digester gas. Iron chloride addition is regarded as the most cost-effective method to control this problem.

Alternatives

Hydrogen Peroxide. Hydrogen peroxide is a very effective and powerful oxidant. An advantage of hydrogen peroxide is that it decomposes into oxygen and water, thus providing dissolved oxygen to help with further prevention of septic conditions. About 15 minutes of contact time is required to achieve the full oxidizing effect of hydrogen peroxide. Since hydrogen peroxide will react with various organic material, the dosage required is greater than the dosage indicated by stoichiometric oxidation of sulfide. In general, hydrogen peroxide to sulfide weight ratios of 2:1 to 6:1 are often required. Hydrogen peroxide typically comes in 50 percent solutions, however, it can produce a highly exothermic reaction at that concentration if accidentally catalyzed. Thirty-five percent solutions are also available, which may be helpful in meeting fire prevention requirements.

Sodium Hypochlorite. Sodium hypochlorite (NaOCl) is also a very powerful oxidant. It acts essentially the same as adding chlorine to wastewater, except that it is considerably safer to deal with than chlorine gas. Sodium hypochlorite is typically delivered in about 13 percent liquid solutions and is relatively easy to handle compared with most chemicals. The chemical reacts almost instantaneously with dissolved sulfide, oxidizing it to sulfate and to elemental sulfur. Sodium hypochlorite also has good performance in oxidizing organic sulfur compounds. NaOCl often requires larger on-site storage tanks than other chemical options because of its relatively low chemical content (i.e., 13 percent). Good mixing of the chemical and the wastewater at the point of injection is important. Dosage ratios by weight (NaOCl to sulfide) are usually between 4:1 and 8:1 for situations similar to the one at Lodi.

Iron Chloride. Iron addition (either ferrous chloride or ferric chloride) at or upstream from the plant headworks could help solve multiple problems at the Lodi plant. The practice of adding iron chloride to upstream sewers to help control both influent plant sulfide as well as digester gas hydrogen sulfide is well proven. This is the basic method practiced by the Los Angeles County Sanitation Districts, as well as plants in Omaha, Nebraska, Wichita, Kansas, Phoenix, Arizona and many others. The precipitation reaction of iron (Fe) with sulfide (S) takes about 15 to 20 minutes for full effect, and, therefore, maximum effectiveness is achieved at the plant headworks by adding the chemical at least this far (about ½ mile) upstream.

There are three major possibilities for the location of iron chloride feed to the system. These are described briefly as follows:

1. Direct addition to digesters. If the only problem to be solved is digester gas hydrogen sulfide control, iron can be added direct to each digester, or possibly to the sludge feed lines. This will require the least dosage rate, and would typically control digester gas hydrogen sulfide down to less than 500 ppm, and probably in the 200 to 300 ppm range, cost-effectively.

2. Iron addition at headworks. Often a more convenient location for iron feed is the headworks rather than into the digesters or sludge piping. The dosage rate may increase slightly because some sulfide in the liquid wastewater will be treated in addition to the primary goal of treating the sulfide in the thickened sludge in the digesters. The iron becomes part of the primary sludge and proceeds to the digester to control sulfide. The chloride becomes part of the effluent to the plant (adds about 20 mg/L of chloride to plant effluent).

3. Iron addition upstream (minimum 20 minutes upstream). This approach treats sulfide in influent wastewater, and, assuming the dosage is sufficient, provides iron in the primary sludge which controls sulfide in the digesters. This approach provides reduction of hydrogen sulfide and odor levels at the plant headworks and PSBs as well as digester sulfide control. Dosage rate is higher because the entire domestic flow stream is treated by the iron in this case. This approach also adds more chloride to the plant effluent.

A typical dose for digester gas hydrogen sulfide control would be about 8 mg/L as $FeCl_3$. Additional dosage to control plant influent dissolved sulfide on an annual average basis is probably about 8 to 12 mg/L as $FeCl_3$. Assuming an influent average combined dose of 18 mg/L at 8.5 mgd flow rate, and a cost of 15 cents per pound of $FeCl_3$ (dry weight), the cost of the chemical is estimated to be $65,000 per year. This cost is associated with adding iron per location No. 3 identified above.

Evaluation of Chemical Addition Alternatives

The advantages, disadvantages, and costs of the chemical addition alternatives are summarized in Table 12-2. Hydrogen peroxide performs well and has been gaining popularity for preventing sulfide and odor generation in sewer trunk lines. Hydrogen peroxide is less costly than sodium hypochlorite. Iron chloride is the lowest cost alternative and has the added benefit of reducing hydrogen sulfide in digester gas.

Table 12-2. Odor Control Alternatives Summary

Process	Alternatives[a]	Advantages	Disadvantages	Capital Cost[b], dollars	Annual O&M Cost, dollars
Chemical Addition for Odor Control	Sodium hypochlorite	High performance	High dose needed, high cost	210,000	125,000
	Hydrogen peroxide	High performance if added 30 minutes upstream	High dose needed, high cost	180,000	115,000
	Iron chloride	Treats inlet sulfide and digester gas H_2S	Hazardous material (pH<2), reduces wastewater alkalinity	160,000	65,000

A disadvantage of adding iron chloride is that it destroys alkalinity. Both $FeCl_2$ and $FeCl_3$ solutions typically have pH of 1 to 1.5 due to strong hydrochloric acid content. Since the Lodi plant already has problems at times with low alkalinity and low effluent pH, adding iron chloride could make this problem somewhat worse.

Overall, Iron Chloride is the recommended chemical addition alternative. Pilot testing should be performed to insure that alkalinity is not decreased below an acceptable level.

SEPTAGE HANDLING

Current direct discharge of septage from trucks to the influent flow makes successful odor control difficult at the plant's front-end processes (headworks and PSBs). As was previously discussed in Section 11, a better arrangement would be to store the septage in tank(s) and discharge the flow to the plant influent over the course of each day, or at least several hours of the day. Septage storage tanks can be designed in a variety of ways to limit odor production and odor emissions. The tanks would be contained and foul air withdrawn from them for treatment.

HEADWORKS AREA FOUL AIR CONTAINMENT

The above-described chemical addition system using iron chloride (at location No. 3) will reduce hydrogen sulfide concentrations in the headworks area, but will not eliminate the need for foul air collection and treatment. Several specific sources need to be contained in the headworks area as described below.

Influent Sewers

The existing arrangement of ventilation at the influent box/channels is the correct approach. At this time, we believe about 2,000 cfm should be adequate to remove the foul air from the influent sewers and prevent the escape of this foul air. This foul air withdrawal rate should keep influent sewers under slight negative pressure.

Septage Receiving

As discussed previously in Section 11, an improved septage receiving (and possible storage) structure with bar screen and washdown pad is envisioned for the future. Foul air should be withdrawn from this receiving structure (and storage tanks, if provided) to insure that most odorants are trapped and not released directly to the atmosphere. The foul air flow rate is estimated to be 500 cfm for the structure itself, and perhaps 1,500 cfm if storage tanks are included.

Headworks Room

This room has, and will continue to have, substantial odor sources, especially with a new screening facility located here. Such a below-grade room should be ventilated with at least 20 air changes per hour and the newly revised ventilation system in this room may meet this criterion. A more detailed evaluation of the ventilation in this room should be undertaken to determine if fresh air supply is reaching all primary work areas. Foul air withdrawal may be improved with new intakes located low, near the floor level. The room should be kept under slightly negative pressure to prevent out-leakage of foul air. The current foul air flow rate of 7,000 cfm is assumed to be adequate at the present time. The detritus/grit tank system is also located in this room, as well as the wet wells for influent pumping of both the industrial and domestic influent streams.

Grit Washing and Grit Storage/Loadout

These sources are sufficiently odorous, especially in summer months, that foul air containment and treatment is necessary. The grit washing equipment can be mostly contained so that foul air is captured for treatment. Likewise, grit storage bins can be covered and flexible duct connections made to such covers for foul air withdrawal. A foul air flow rate of 2,000 cfm is estimated at this time to be required from these sources. In the future, the grit storage/loadout could include dewatered screenings as well, which would bring additional odorous material to this system.

Scum/Grease Sump

This sump can be covered rather easily, although corrosion protection for the walls may also be necessary. Foul air withdrawal is estimated to be 500 cfm. A connection port is needed to allow grease discharge from trucks to this sump.

The above foul air sources should be combined with ductwork and fan(s) for one foul air treatment system. The combined system is desirable so that peak emissions from each source will be diluted by other sources, thus minimizing the peak impact of any given source. The elements of the foul airstreams are listed in Table 12-2 along with estimated hydrogen sulfide concentrations. Two scenarios are provided: the first set of columns shows expected concentrations if no iron is fed at upstream locations; and the second set of columns showing expected concentrations if iron is fed at least 20 minutes upstream of the headworks. The differences show that there is some advantage from upstream chemical feed as far as headworks odor is concerned—this difference could be important concerning the degree of foul air treatment required on this combined airstream. Another primary benefit of upstream iron addition is reduced PSB weir emissions. Prior to actual implementation of this system, a more detailed examination of required airflow rates and characteristics of the foul air should be confirmed through on-site tests.

FOUL AIR TREATMENT FOR HEADWORKS AREA SOURCES

The combined foul air from the sources (Table 12-3) is a relatively typical foul airstream from headworks-type sources at wastewater plants. There could be significant hydrogen sulfide peaks (as indicated in the table), but also peaks that would contain significant odor strength from organic compounds associated with raw wastewater, septage, grease/scum, and septic food processing wastes. The three most common methods of foul air treatment for this type of foul air are packed bed wet scrubbers, activated carbon adsorption, and bulk media biofiltration.

Table 12-3. Estimated Foul Air Characteristics (Headworks and Related Sources)

Foul Air Source	Estimated Foul Air Flow Rate, cfm	No Iron Addition Upstream		With Iron Addition Upstream	
		Ave H_2S, ppm	Peak H_2S, ppm	Ave H_2S, ppm	Peak H_2S, ppm
1. Influent Sewer Box and Channels	2,000	4	40	2	25
2. Septage Discharge (and possible storage tanks)	500 to 1,500	0.1	10	0.1	10
3. Headworks Room	7,000	0.5	5	0.3	2
4. Grit Washing and Loadout	2,000	0.2	5	0.1	2
5. Scum/Grease Sump	500	1	50	1	50
Total and Composites	12,000 to 13,000	1	10[a]	0.6	6[a]

[a] Composites for peak H_2S not strictly mathematical.

EVALUATION OF FOUL AIR TREATMENT ALTERNATIVES

The costs and subjective considerations for the foul air treatment alternatives are summarized in Table 12-4. We believe packed bed wet scrubbers should be eliminated at the outset as they are the highest cost method and have questionable performance on foul air with the characteristics identified. The other two options – activated carbon adsorption and bulk media biofiltration – can both provide a high level of treatment and their overall costs are quite similar. The main difference between these two methods is space requirements. Required space for a biofilter is shown to be 4,000 to 5,000 square feet for the sources identified here. In actuality, more space than this should be set aside for long-term foul air treatment needs, since the foul air system may need to be expanded in the future with new or different headworks facilities and additional sources in this area that may, with time, need to be treated. Probably 7,000 to 10,000 square feet should be set aside for eventual biofiltration treatment facilities. When sufficient space is not available near the headworks, then implementation of activated carbon is likely to be more attractive. Both of these two methods (carbon and bulk media biofilters) would have better odor control performance and slightly reduced operating costs with the option of iron chloride fed upstream, rather than iron chloride fed at the headworks.

Based on the planned facilities layout (Section 11, Figure 11-4), there is sufficient room for bulk media biofilters to the west of the digesters. Therefore biofilters are the recommended alternative for foul air treatment. A new foul air ductwork and fan system will be required to transport these foul airstreams to the treatment system. The costs for this system are not included in the costs shown in Table 12-4 because they are common to all alternatives. Total system costs are included in Section 13.

Table 12-4. Advantages and Disadvantages of Foul Air Treatment Options (Lodi Headworks System)

Optional System with Costs[a]	Advantages	Disadvantages
Packed Bed Wet Scrubbing (w/ NaOH and NaOCl chemicals) Capital Cost = \$460,000 O&M Cost = \$50,000/yr.	• Common technology • Small footprint • Can handle particulates easily • Handles rapid increase in H_2S concentration	• Organic pollutant removal is poor • Relatively high labor needs • Residual bleach odor downwind at times • Chemicals need special handling
Activated Carbon Adsorption Capital Cost = \$560,000 O&M Cost = \$40,000/yr.	• Common technology • Small footprint • Organic odorants and H_2S removal • Simple operation under normal conditions • Also helps to control VOC emissions	• Carbon regeneration or replacement can be complicated • Spent carbon disposal can be problem • Higher gas pressure needed • Carbon can be used rapidly at high pollutant concentrations • Filters required to eliminate particulates in foul air
Bulk Media Biofiltration Capital Cost = \$480,000 O&M Cost = \$35,000/yr.	• Biological technology more operator-friendly • Requires no chemicals, normally • Treats wide variety of odorants • Also controls some VOCs	• Large footprint (4,000 to 5,000 ft^2 minimum) • Little ability for process control • Media replacement likely every 4 to 5 years • Filters required to eliminate particulates in foul air • Relatively high gas pressure required with monitoring

[a] Capital costs shown include 20% for engineering and 40% contingency.
Headworks, ductwork, and fan system costs (not shown) are estimated at $60,000.

SUMMARY OF ODOR CONTROL RECOMMENDATIONS

The odor control alternatives evaluated are summarized in Table 12-5. Recommended improvements are summarized in Table 12-6.

Table 12-5. Odor Control Alternatives Summary

Process	Alternatives[a]	Advantages	Disadvantages	Capital Cost[b], dollars	Annual O&M Cost, dollars
Chemical Addition for Odor Control	Sodium hypochlorite	High performance	High dose needed, high cost	210,000	125,000
	Hydrogen peroxide	High performance if added 30 minutes upstream	High dose needed, high cost	180,000	115,000
	Iron chloride	Treats inlet sulfide and digester gas H_2S	Hazardous material (pH<2), reduces wastewater alkalinity	160,000	65,000
Headworks Foul Air Treatment	Chemical scrubbing	Common technology, small footprint	Chemical handling	460,000	50,000
	Carbon adsorption	Single operation, small footprint	Carbon can be used up rapidly with strong odor	560,000	40,000
	Bulk media biofiltration	Technology is operator-friendly	Space requirements	480,000	35,000

(a) Alternatives shown in bold are recommended.
(b) Includes 20% for engineering and 40% contingency.

Table 12-6. Recommended Improvements

Process	Deficiencies	Recommend Improvements
Headworks and Digesters	High gas-phase H_2S causing odor/corrosion at headworks. Digester gas H_2S must meet air permit limits.	Add iron chloride to upstream sewer(s).
Headworks and Related Units	Strong odor impacts offsite from these odor sources.	Improve foul air capture and ventilation. Treat foul air using bulk media biofiltration.

SECTION 13. EVALUATION OF COMBINED TREATMENT AND DISPOSAL/REUSE ALTERNATIVES

DESCRIPTION OF ALTERNATIVES

The combined treatment and disposal/reuse alternatives were described in detail in Section 5. Summary descriptions of the alternatives and their most significant unique components are listed in Table 13-1.

Table 13-1. Unique Components of Major Alternatives

Alternative	Description	Unique Components
DC-D	Discharge to Dredger Cut	Tertiary Treatment 500 af Additional Storage Aggressive Source Control
DC-W	Discharge to Dredger Cut with wetlands for polishing treatment and storage	Tertiary Treatment Wetlands 250 af Additional Storage
BC-D	Discharge to Bishop Cut through an outfall pipeline	Outfall Pipeline Tertiary Treatment Source Control
BC-W	Discharge to Bishop Cut through an outfall wetlands	Outfall Wetlands Tertiary Treatment
BC-PD	Partial discharge to Bishop Cut through an outfall pipeline, partial percolation disposal	Outfall Pipeline Source Control Denitrification Percolation Basins/Fields
BC-PW	Partial discharge to Bishop Cut through an outfall wetlands, partial percolation disposal	Outfall Wetlands Denitrification Percolation Basins/Fields
LD	Land discharge – complete effluent disposal through percolation and agricultural irrigation reuse	Percolation Basins/Fields Denitrification

EVALUATION OF ALTERNATIVES FOR DENITRIFICATION

Three of the seven alternatives previously listed rely on percolation disposal or land application of treated effluent. These alternatives may require some level of effluent denitrification. This can be accomplished either using wetlands as was discussed in Section 7 or using conventional anoxic denitrification in the secondary biological treatment process as was discussed in

Section 11. Selecting a preferred denitrification alternative first simplifies the comparison of combined treatment and disposal/reuse alternatives in this section of the Master Plan.

The costs of denitrification for the full 8.5 Mgd Master Plan flow are shown in Table 13-2. As discussed in Sections 7 and 10, to remove over 40 percent of the nitrogen from a fully nitrified effluent would require approximately 60 acres of wetlands. Conventional denitrification in the secondary biological process would require two additional tanks, baffles, recycle pumps, mixers, and miscellaneous related facilities. Conventional denitrification could easily achieve 40 percent nitrogen removal without an external source of carbon.

Table 13-2. Comparison of Estimated Costs for Denitrification Alternatives

Denitrification Method	Capital Cost	O&M	Life Cycle
Wetlands (60 ac)	1,800,000	110,000	3,000,000
Secondary Process	5,800,000	100,000	6,900,000

Wetlands are considerably more economical than conventional denitrification in the secondary biological process. Wetlands provide some additional benefits over conventional denitrification such as TSS removal, storage, temperature adjustment, and wildlife habitat. Based on the substantial cost differential and the other benefits, wetlands are clearly the preferred method for denitrification.

COSTS OF COMBINED TREATMENT AND DISPOSAL/REUSE ALTERNATIVES

Costs of Alternatives

Cost estimates for the unique components of the major alternatives are shown in Table 13-3. Costs for additional recommended general improvements to the treatment and disposal/reuse facilities which are common to all alternatives and which will be required are not included in Table 13-3 costs. These common costs were summarized previously Table 11-13. The costs of percolation basins shown in Table 13-3 are based on assuming that the basins could be relatively simple with low containment berms and only a few widely spaced underdrains for groundwater level control.

There are a couple interesting comparisons evident in Table 13-3. First, a 40-acre outfall wetlands to Bishop Cut would cost less than an outfall pipeline. A 60-acre outfall wetlands would cost about the same as an outfall pipeline. Second, the cost of the additional effluent storage needed for the Dredger Cut discharge alternatives make those alternatives more expensive than the Bishop Cut discharge alternatives.

The only practical method for reducing effluent concentrations of heavy metals for alternatives without wetlands is source control. The capital and O&M costs shown in Table 13-3 for source

Table 13-3. Costs for Unique Components of Major Alternatives

Alternative	Facility	Capital Costs, $	Annual O&M, $	Life Cycle Cost, $
DC-D	500 af Storage	3,800,000	108,000	4,944,000
	Source Control	500,000	100,000	1,559,000
	Expanded Chlorine Contact Tank	1,070,000	10,000	1,176,000
	Granular Media Filtration Facilities	11,910,000	392,000	16,063,000
	Alternative DC-D Totals	17,280,000	610,000	23,742,000
DC-W	130 ac Wetlands	3,900,000	182,000	5,828,000
	250 af Storage	1,900,000	54,000	2,472,000
	Expanded Chlorine Contact Tank	1,070,000	10,000	1,176,000
	Granular Media Filtration Facilities	11,910,000	392,000	16,063,000
	Alternative DC-W Totals	18,780,000	638,000	25,539,000
BC-D	7500 ft Outfall (48")	2,080,000	24,000	2,334,000
	Reaeration, Diffuser	1,100,000	66,000	1,799,000
	Source Control	500,000	100,000	1,559,000
	Expanded Chlorine Contact Tank	1,070,000	10,000	1,176,000
	Granular Media Filtration Facilities	11,910,000	392,000	16,063,000
	Alternative BC-D Totals	16,660,000	592,000	22,931,000
BC-W	100 ac Wetlands	3,000,000	150,000	4,589,000
	Reaeration, Diffuser	1,200,000	66,000	1,899,000
	Expanded Chlorine Contact Tank	1,070,000	10,000	1,176,000
	Granular Media Filtration Facilities	11,910,000	392,000	16,063,000
	Alternative BC-W Totals	17,180,000	618,000	23,727,000
BC-PD	7500 ft Outfall (42")	1,850,000	24,000	2,104,000
	Reaeration, Diffuser	1,100,000	66,000	1,799,000
	Source Control	500,000	100,000	1,559,000
	260 ac Percolation	10,140,000	100,000	11,199,000
	Denitrification Wetlands (40 ac)	1,200,000	80,000	2,048,000
	Alternative BC-PD Totals	14,790,000	370,000	18,709,000
BC-PW	Denitrification Wetlands (40 ac)	1,200,000	80,000	2,048,000
	Outfall Wetlands (60 ac)	1,800,000	90,000	2,753,000
	Reaeration, Diffuser	1,100,000	66,000	1,799,000
	260 ac Percolation	10,140,000	100,000	11,199,000
	Alternative BC-PW Totals	14,240,000	336,000	17,799,000
LD	400 ac Percolation	15,600,000	180,000	17,507,000
	Denitrification Wetlands (60 ac)	1,800,000	100,000	2,859,000
	Alternative LD Totals	17,400,000	280,000	20,366,000

Notes:
1. All costs are based on ENR 7000.
2. Wetlands and percolation disposal costs include estimated costs for purchasing farm land at $15,000/ac. for percolation areas and $6,000 per acre for all other areas.

control of heavy metals, and the level of success such a program could have, are no more than a rough guess. A detailed source study would be required to determine where the heavy metals are coming from and what could be done to reduce the concentrations. Then more accurate cost estimates for source control could be developed.

COSTS OF ALTERNATIVES WITH SPORTS COMPLEX

The Sports Complex requires a 2.5 Mgd supply of tertiary treated effluent for irrigation. If the Sports Complex is constructed, it will also reduce the amount of land available for application of industrial effluent and biosolids to 490 acres. Since 700 acres of land is recommended for biosolids and industrial effluent (see Section 10), construction of the Sports Complex would leave a deficit of 210 acres. This deficit would need to be satisfied by the purchase of additional nearby land. For the purposes of this evaluation, it was assumed that the costs of 2.5 Mgd of tertiary treatment capacity and 210 additional irrigated acres would be borne by the Sports Complex project rather than be financed through sewer rates.

The allocation of 2.5 Mgd out of the 10.0 Mgd of tertiary treatment capacity (filtration facilities and chlorine contact tank capacity) to the Sports Complex project cost results in a effective capital cost reduction of $4 million for alternatives DC-D, DC-W, BC-D, and BC-W. Having 2.5 Mgd of tertiary treated water which could be discharged during the winter rather than disposed of through percolation would reduce the required percolation areas by about 110 acres for alternatives BC-PD and BC-PW. This would reduce capital costs for alternatives BC-PD and BC-PW by approximately $4.4 million. Having tertiary treatment capacity available would not affect alternative LD (land discharge) since the same amount of percolation basins and fields would be required. The total capital, O&M, and life cycle costs for all the alternatives if the Sports Complex is constructed are shown in Table 13-4.

Table 13-4. Costs of Major Alternatives Assuming Sports Complex Funds 2.5 Mgd Tertiary Treatment Capacity

Alternative	Capital Cost, million dollars	Annual O&M, million dollars	Life Cycle Cost, million dollars
DC-D	13.3	0.527	18.9
DC-W	14.8	0.555	20.7
BC-D	12.7	0.509	17.9
BC-W	13.2	0.535	18.9
BC-PD	8.4	0.360	11.8
BC-PW	10.1	0.306	13.3
LD	17.8	0.280	20.8

SUBJECTIVE CRITERIA

The combined treatment and disposal/reuse alternatives were compared based on the subjective criteria presented in Section 6. The results of that evaluation are summarized in Table 13-5 and discussed in the following paragraphs.

Compliance with Future Discharge Requirements

Alternatives DC-D was not rated highly because there are potential problems with low dissolved oxygen and low dilution in Dredger Cut which may only be partially addressed by tertiary treatment and extra storage. In addition, algae regrowth in storage reservoirs will probably significantly increase the BOD and TSS of stored effluent beyond the anticipated limit of 10 mg/L. Alternative DC-W provides improved trace metals removal and prevents algae regrowth when compared to Alternative DC-D, and was therefore given a moderate ranking. Alternative BC-D would provide substantially increased dilution, although future discharge requirements related to trace toxins and temperature could still be problematic. Alternative BC-W is rated highly because of the dilution in Bishop Cut and the polishing treatment provided by the wetlands. Alternatives BC-PD and BC-PW would reliably meet future discharge requirements provided that there continues to be adequate dilution in Bishop Cut to avoid tertiary treatment for that portion of the effluent discharged to Bishop Cut. They were only given moderate ratings because of the potential for less dilution in Bishop Cut if the proposed CalFed Delta channel modifications are constructed. Alternative LD would greatly reduce the stringency and complexity of discharge requirements by eliminating discharge to surface waters.

Reliability

Alternative BC-D would be straightforward and predictable, resulting in a high reliability rating. Alternative BC-W would be nearly as good as BC-D, except that the wetlands could reduce the predictability of the effluent BOD and TSS. Alternatives BC-PD, BC-PW, and LD utilize simpler processes and therefore should be relatively reliable.

Flexibility

Alternatives DC-D and DC-W depend entirely on discharge to Dredger Cut with the accompanying most challenging discharge requirements. This results in relatively low flexibility ratings. Alternatives BC-D and BC-W depend upon discharge to Bishop Cut with easier discharge requirements, therefore, these were given moderate flexibility ratings. The other alternatives allow more flexibility in discharge location and level of treatment. Alternatives BC-PD and BC-PW would provide facilities for both discharge to Bishop Cut and percolation disposal, resulting in a high level of flexibility.

Ease of Operation and Maintenance

None of the alternatives are particularly difficult to operate. Wetlands operation and maintenance were discussed in Section 7. Percolation fields or basins would need to be disked on a regular basis to maintain high infiltration rates. Tertiary treatment would add some challenges in terms of operation and maintenance.

Table 13-5. Subjective Comparison of Alternatives

Subjective Criteria	Weightings	Alternatives						
		DC-D	DC-W	BC-D	BC-W	BC-PD	BC-PW	ZD
Compliance with Future Discharge Requirements	1.5	1	2	3	4	3	3	5
Reliability	1.5	3	2	4	3	4	4	4
Flexibility	1.5	1	2	3	3	4	4	3
Ease of Operation and Maintenance	1	3	3	3	3	4	4	4
Ease of Implementation	1	4	3	3	3	3	3	2
Environmental Impacts	1	3	4	3	5	3	4	4
Safety	1	4	4	4	4	4	4	5
Open Space/Recreational Benefits	0.5	1	4	1	4	2	3	2
Aesthetics	0.5	3	5	3	5	3	4	2
Secondary Economic Benefits	0.5	1	2	1	2	1	2	1
Resource Management Considerations	0.5	4	4	4	4	2	3	2
Totals		28	35	32	40	33	38	34
Totals (weighted)		26	30.5	32.5	37.5	34.5	37.5	36.5

Ease of Implementation

Alternative LD would be considered difficult to implement rapidly because of the large additional land requirements and institutional considerations. Alternatives BC-PD, BC-PW, BC-W, and DC-W would require some additional land and therefore were only given moderate ratings.

Environmental Impacts

No significant adverse environmental impacts are anticipated with any of the alternatives. The wetlands alternatives (DC-W, BC-W, BC-PW) would actually provide additional positive environmental benefits.

Safety

All the alternatives are relatively safe. Alternative LD would involve the lowest use of chlorine and the least potential for accidents in the discharge waters.

Open Space/Recreational Benefits

The wetlands alternatives would provide substantial open space and recreational benefits in terms of bird and wildlife viewing.

Aesthetics

The wetlands alternatives would have excellent aesthetics. The land discharge alternative was rated moderately low because there would be a large continuous area with disked, bare fields and basins. Other alternatives had moderate ratings.

Secondary Economic Benefits

All of the alternatives would reduce secondary economic benefits compared with the current facilities because the amount of farmland in production in the area would be reduced. Construction of the Sports Complex would substantially increase secondary economic benefits, but the Sports Complex could be an adjunct to any of these alternatives.

Resource Management Considerations

Alternatives with tertiary treatment (DC-D, DC-W, BC-D, and BC-W) would provide the best potential for increased wastewater reclamation because the effluent could be for unrestricted agricultural or landscape irrigation.

COMPARISON OF ALTERNATIVES

A graphical comparison of the major alternatives is presented in Figure 13-1. The overall lowest cost alternative is BC-PW, closely followed by BC-PD and LD. The lower relative costs of these alternatives are primarily due to the lack of expensive tertiary treatment facilities. The reason that Alternative BC-PW is less costly than Alternative BC-PD is that the cost of a small outfall wetlands is lower than the cost of an outfall pipeline. Alternative BC-PW has a relatively high subjective rating compared with the other alternatives. The weakness of Alternative BC-PW is that future changes to the Delta channels could force the City to construct tertiary treatment for about 40 percent of the effluent flow or to convert entirely to the land discharge alternative. Constructing tertiary treatment facilities for 40% of the effluent flow would cost about $6 million. The transition to LD would result in the abandonment of approximately $2 million worth of facilities constructed for discharge to Bishop Cut plus cost an extra $5 million for more percolation basins.

Alternative LD is very intriguing because it could greatly simplify future treatment and monitoring requirements. However, Alternative LD is also the most difficult alternative for which to estimate costs and performance. For example, the costs of percolation basins shown in Table 13-2 assume relatively basic basin construction with small containment berms. The costs of farm fields converted for dead level basin winter irrigation would be significantly less than the costs shown in Table 13-2, while the costs of conventional high rate rapid infiltration facilities would be slightly higher than those shown in Table 13-2. It may also be acceptable to have ammonia in the effluent if good nitrification and denitrification can be achieved in the percolation basins soil profile. This would eliminate the need for additional aeration basins and denitrification wetlands.

The significant unknowns with Alternative LD are the availability of adequate land with highly permeable soil, the response of the water table, and the nitrogen removal performance. The 400-acre estimated land requirement was based on the soils testing data and initial groundwater mounding calculations. Pilot testing and additional discussions with landowners are needed to adequately assess the actual costs and feasibility of Alternative LD. This can also be said for the percolation components of Alternatives BC-PD and BC-PW, although the smaller scale of these alternatives would make them easier to implement with less impact on the water table.

Alternative BC-W has the highest subjective rating and a moderately high overall cost. If percolation disposal became too difficult to implement because of land acquisition or groundwater level control considerations, it would be the preferred alternative. Alternative BC-D is less expensive than BC-W, but not by enough to justify the loss of benefits from wetlands.

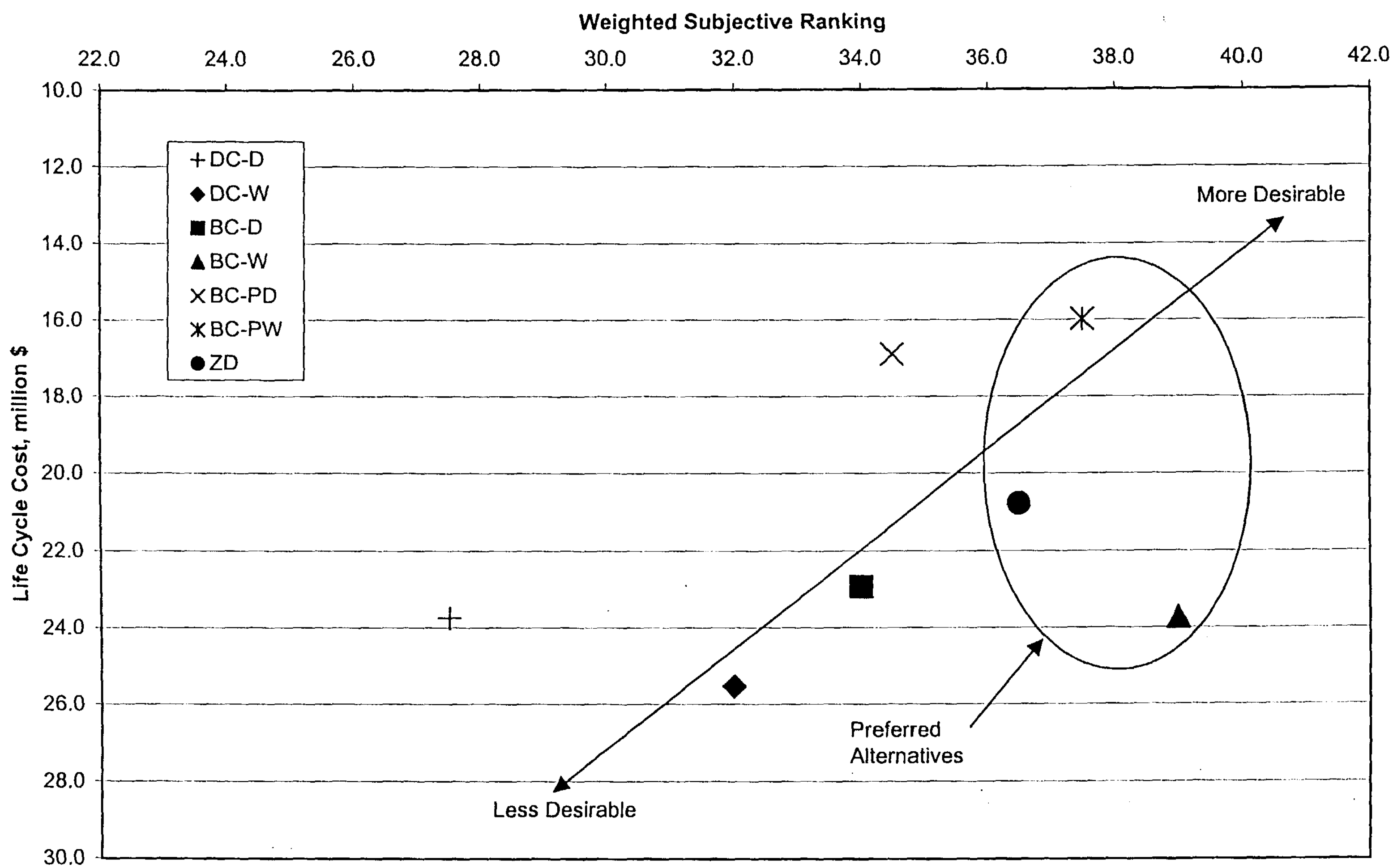
The Dredger Cut discharge alternatives are relatively expensive to implement because of the need for additional storage. They also have relatively low subjective ratings because of the greater challenges in reliably meeting Year 2004 and future discharge requirements.

RECOMMENDATIONS

Based on current assumptions, and considering cost, Alternatives BC-PW and LD are the preferred alternatives. Selecting between these alternatives (and possibly other alternatives) would be dependent on performance of percolation disposal facilities and requirements for discharge to Bishop Cut.

A stepwise approach is needed to arrive at the optimum wastewater treatment and disposal/reuse facilities. The recommended initial actions are as follows:

Figure 13-1. Comparison of Major Alternatives



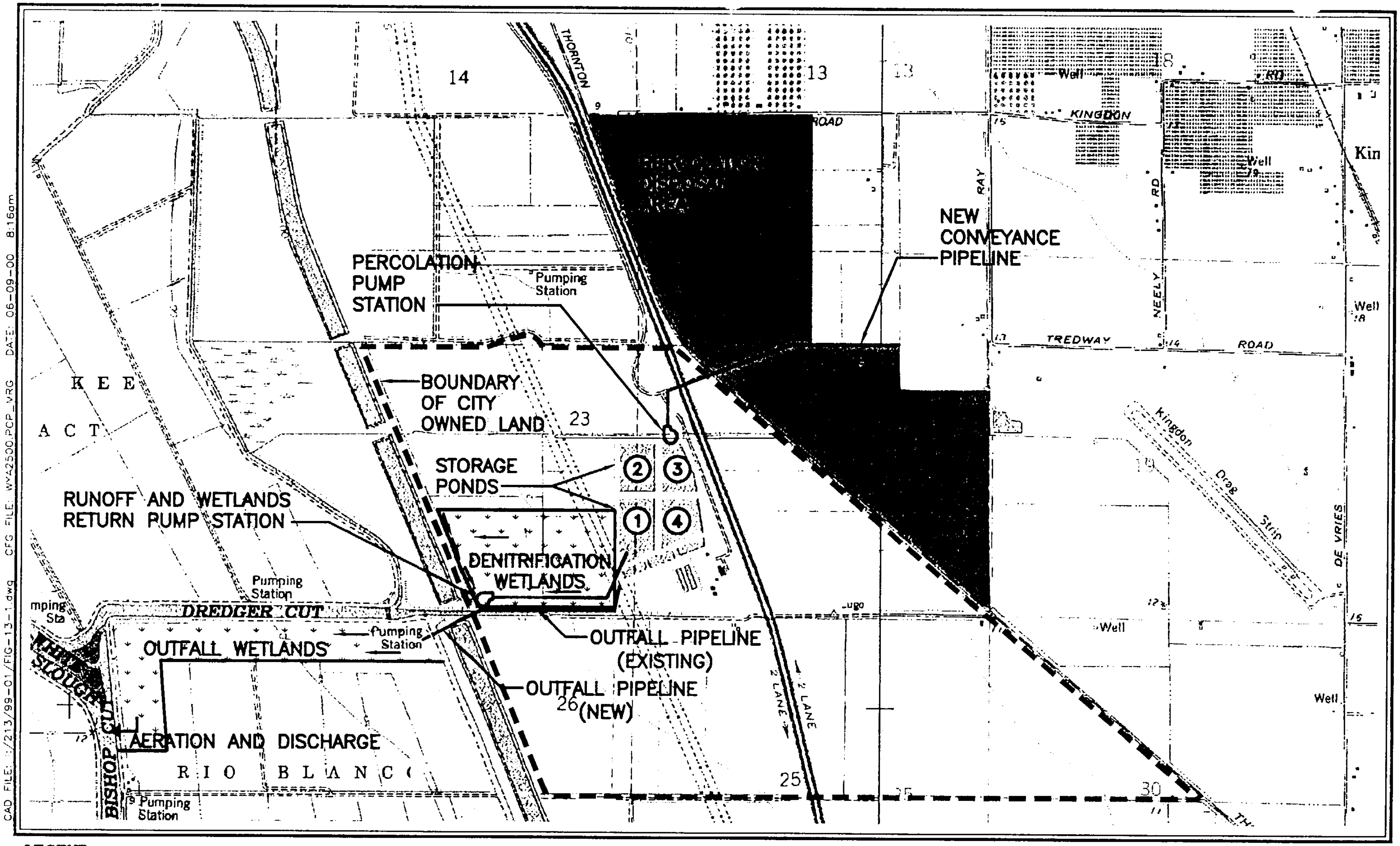
- Perform pilot testing of winter percolation disposal at full field scale during late 2000 and early 2001.

- Submit an application to the Regional Water Quality Control Board for a new discharge permit for both discharge to Bishop Cut and winter percolation. This should help confirm whether or not the Master Plan assumptions for anticipated discharge requirements are correct.

- Begin discussions with nearby landowners about purchasing or otherwise obtaining operating control of sufficient area for the LD or BC-PW alternatives.

If the results of the percolation disposal pilot testing and discussions with landowners are positive, the cost comparisons of the LD and BC-PW alternatives should be updated. The City should then decide which of the two alternatives to pursue based on costs and land availability. If the results of the percolation disposal pilot testing and landowner discussions are negative, the City should pursue the BC-W alternative. The conceptual layouts for the BC-PW alternative is shown in Figure 13-2. Potential winter percolation areas for the BC-PW and LD alternatives were shown previously in Figure 10-1. The BC-W alternative was shown previously in Figure 7-2.

Detailed facilities to accompany each preferred alternative along with other recommended general improvements are presented in Section 14.



LEGEND:

Boundary of City–Owned Land

Constructed Wetlands

Percolation Disposal Area

N

0 1000 2000

SCALE IN FEET

Figure 13-2

City of Lodi
Wastewater Master Plan

Alternative BC - PW
Conceptual Layout

WEST YOST & ASSOCIATES

DRAFT

SECTION 14. RECOMMENDED WASTEWATER PROGRAM

pending